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October 1982

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**Federal Highway
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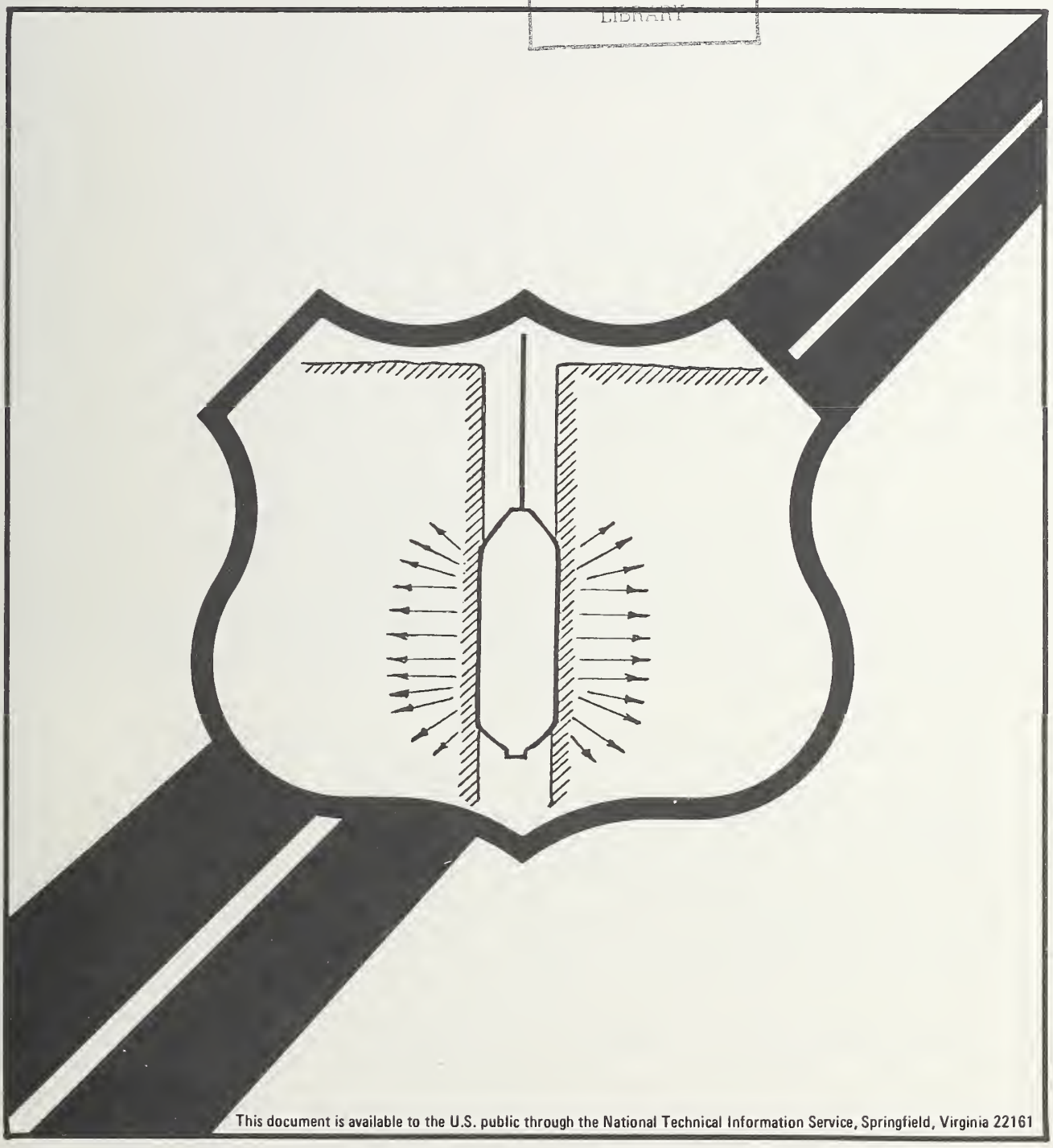

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FOREWORD

This is the second in a series of five reports on in situ testing of soils. The series is intended to explore and summarize the present state of knowledge on in situ testing as it applies to the design of underground structures. The other reports are listed on the opposite page.

This report was prepared by Soil Testing Services, Inc., now called STS Consultants, Ltd., and should be useful to geotechnical, structural, and other civil engineers involved in the design and construction of underground structures.

Copies of the report are being distributed by FHWA transmittal memorandum. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Richard E. Hay, Director
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15. Supplementary Notes

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16. Abstract

The Menard pressuremeter is an in situ testing device which can be used to provide design parameters for nearly all soil and soft rock formations. Currently, its use is rather limited due to difficulties in obtaining valid results, primarily due to inexperience of the user and lack of information regarding U.S. evaluation methods. This report discusses some U.S. testing experiences, prime uses of the equipment, its limitations and methods of evaluation. In addition, nine case histories are included to point out typical usage and conditions where the pressuremeter was an appropriate, in situ measuring device.

This is the second of five volumes. The report numbers and subtitles of the other volumes are: Volume 1, FHWA/RD-81/109, Subtitle: "Bore Hole Shear, Earth Settlement and Earth Penetrometer Probes; Volume 3, FHWA/RD-81/111, Subtitle: "Vane Shear and Cone Piezometer;" Volume 4, FHWA/RD-81/112, Subtitle: "Static Penetrometers;" Volume 5, FHWA/RD-81/113, Subtitle: "Dutch Cone Penetrometer."

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PREFACE

This report is based upon the writers' experience and material from the files of the Soil Testing Services, Inc. corporate office in Chicago, Illinois and several of the affiliate office files. The data used in preparation of the report has been accumulated since 1970, when the first STS pressuremeter was obtained. The results of several different types of pressuremeters, on over several hundred projects, have been used in the report preparation. Experience gained in actual field testing and office engineering evaluation is presented in the form of test summaries and case histories.

The writers hope that the guidelines presented herein will result in improved pressuremeter usage and more accurate field testing procedures, so that wider use of this in situ testing device will result.

Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
	AREA			
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
	MASS (weight)			
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
	VOLUME			
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
	AREA			
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
	MASS (weight)			
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
	VOLUME			
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F

METRIC CONVERSION FACTORS

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LIST OF SYMBOLS

AX	Drilling & testing equipment with 1.9 in. (4.6cm) width
c	Cohesion
c_a	Adhesion between foundation & soil
BX	Drilling & testing equipment with 2-3/8 in. (6.0cm) width
D	Depth of embedment
D_c	Consolidation test reload modulus
E	Menard modulus, can refer to E_d, E or E^+, though typically refers to E_d
E_d	Deformation modulus that controls distortion settlement
E	Rebound modulus
E^+	Recompression modulus
G	Shear modulus
h	Depth
k	Modulus of soil reaction
K	Bearing capacity factor
K_β	Shear Strength Factor
K_o	Coefficient of earth pressure at-rest
L	Length of the footing
N	Standard penetration test resistance
NX	Drilling & testing equipment with 3.0 in. (7.6cm) width
PMT	Acronym for "pressuremeter test"
P	Mean normal stress applied to soil by foundation
P_c	Preconsolidation pressure
P_f	Pressure at which the soil fails, from pressuremeter test
P_l	Theoretical limit pressure at infinite expansion of the cavity, from pressuremeter test
P_o	Horizontal earth pressure
q	Average bearing stress
Q	Pressuremeter probe size - relationship
q_a	Allowable bearing capacity
q_o	Total stress vertical pressure of the overburden at founding level
Qu	Unconfined compression test strength
q_{ult}	Ultimate bearing capacity

LIST OF SYMBOLS (continued)

R	Half-width or radius of a foundation
R_o	Reference length equals 1 ft. (30cm)
S_o	Shear strength
W	Settlement
W_2	Deviatoric settlement
W_3	Spherical settlement
α	Rheological factor based on soil type
γ	Unit weight of soil
γ_ω	Unit weight of water
ϵ	Strain
λ_2	Foundation shape factor
λ_3	Foundation shape factor
ν	Poisson's Ratio
μ	Volumetric strain

IN-SITU TESTING WITH THE MENARD PRESSUREMETER

I. INTRODUCTION

The pressuremeter is a bore hole device which has been used many years in Europe to measure in-situ properties of soil and rock. Since 1970 it has become increasingly popular in the United States to determine in-situ ground conditions. The utilization of the test results are based upon emperically modified theoretical considerations and also upon "test specific" field measurements.

Louis Menard pioneered the use of the pressuremeter in France and the most widely used pressuremeter in the United States is the Menard type. Other basic models and modifications to the Menard probe have been developed and are becoming more popular in use. In particular, two self-boring pressuremeters have been developed. These are the Cambridge In-situ Probe, developed in England and the Baguelin, or PAF self-boring pressuremeter. The self-boring pressuremeters have been developed due to the limitations of the Menard type for use in very soft soils.

The intent of this report is to aid the pressuremeter user by discussing some U.S. testing experiences, prime uses of the equipment, its limitations and methods of evaluation. In addition, several case histories are included to point out typical usage and conditions where the pressuremeter was an appropriate, in-situ measuring device.

Background

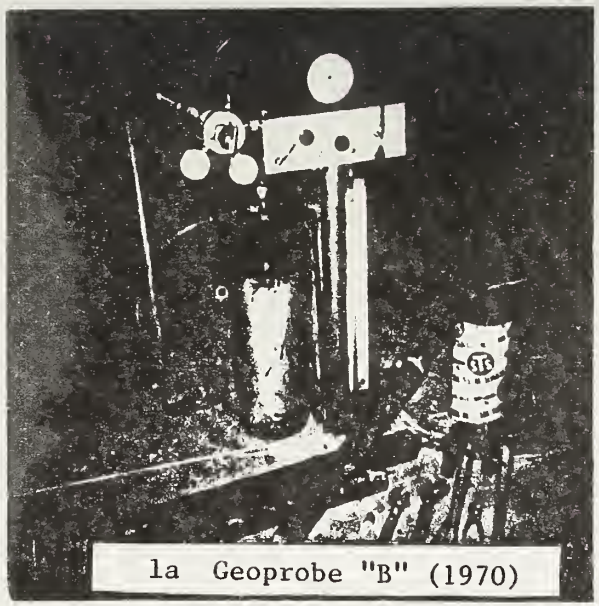
Due to franchise limitations, Menard pressuremeters have not been readily available in the United States until recently. The first unit that our firm used was a Geoprobe Model 2B, purchased in 1970 for $6,000.00. The Geoprobe is shown in Figure 1a. It was quite basic, but also quite sturdy. Modifications, including new valves, tubing and pressure gages have been made, but the unit is still in use (1978).

A Menard type E pressuremeter, shown in Figure 1b, became available for use through a research project with Northwestern University, our firm and the Menard Organization. As indicated by the snow in Figure 1b,the Menard meter is adaptable to all weather conditions. It is more versatile than the Geoprobe model, although the pressure range is not as high.

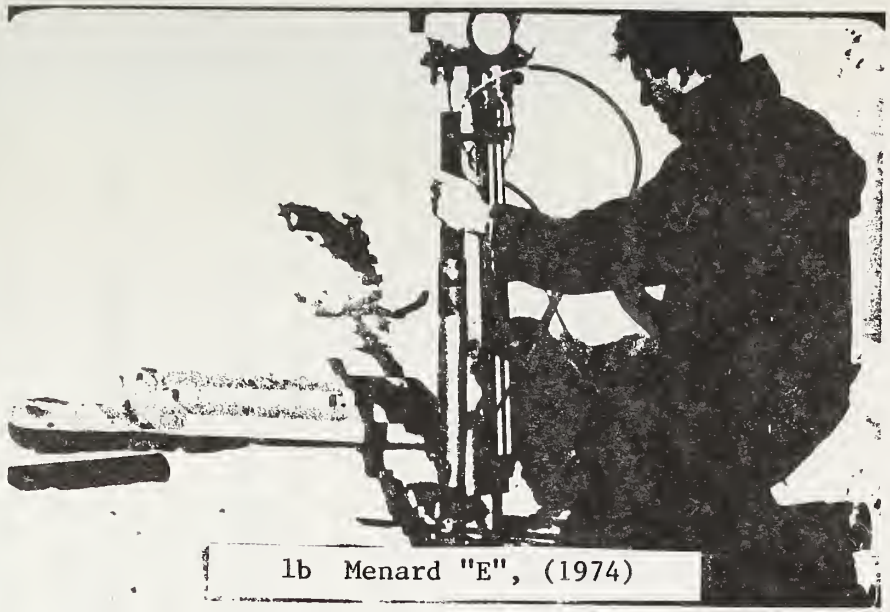
Significant modifications and changes have been developed within the last several years, allowing for more accurate test results and better pressuremeter performance. These include higher pressure ranges, more sensitive valves, and pressure differential systems. Also, the control pannels have been modified so that they fit in a "suitcase" type of container and can be easily transported by plane. Figure 1c shows a model handeled by Roctest, which has been modified for specific applications used by our firm.

Use of various sizes and styles of probes has been evaluated. Figure 1a shows an NX and BX size non-shielded membrane probe. Figure 1d indicates a BX size Menard shielded probe, which we have standardized for use throughout our organization.

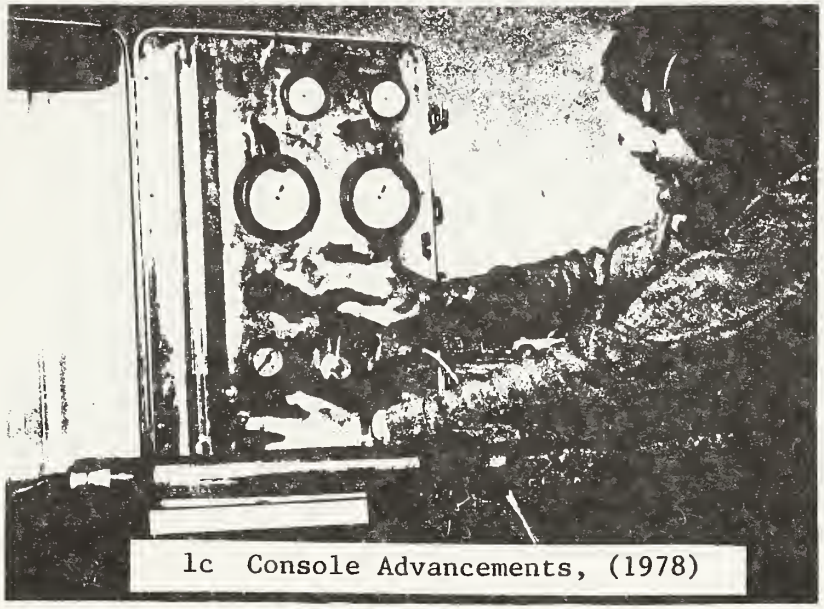
Current (1978) prices for the basic Menard pressuremeter system ranges from $12,000 to $15,000.



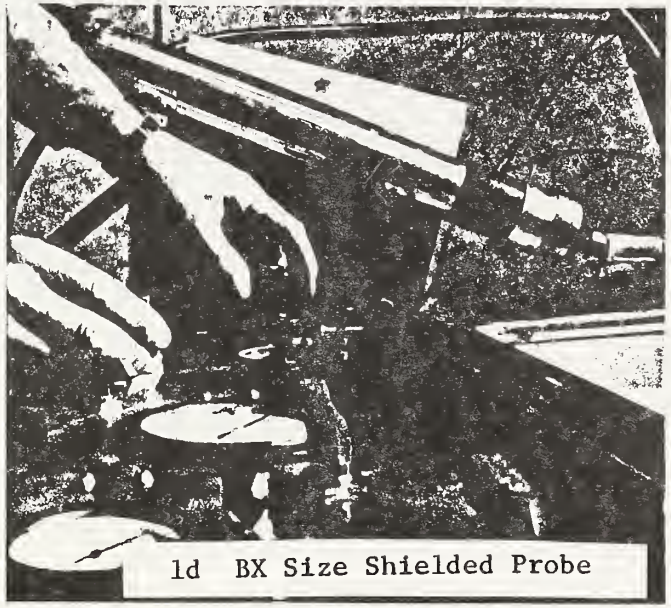
1a Geoprobe "B" (1970)



1b Menard "E", (1974)



1c Console Advancements, (1978)



1d BX Size Shielded Probe

Figure 1 Menard Type Pressuremeters

Pressuremeter tests are performed in predrilled bore holes. The probe, which has three chambers, is lowered into the bore hole to a prescribed depth and then the probe is expanded radially against the sides of the bore hole. Both gas and fluid pressures are typically used for the expansion, although several pressuremeter models use only fluids. The top and bottom chambers, called guard cells, tend to seal off the bore hole, allowing the central chamber or cell to expand in a nearly plane stress/plane strain condition. The pressure in the guard cell is maintained just slightly lower than the central cell pressure. Volume changes in the central cell are measured versus the pressure increment and, in effect, a horizontal stress/strain test curve can be generated.

The results of pressuremeter tests are displayed graphically for pressure versus volume change at each pressure increment. Tests can be taken to near failure of the soil. The parameters obtained from the test curves are used throughout this report, and are briefly described as follows:

Limit pressure, P_l, is the pressure at which failure occurs for the soil. If the soil does not fail, there are several techniques to extrapolate the available data to estimate the limit pressure.

Pressuremeter modulus of deformation, E_d, is a distortion modulus of the soil measured in the "pseudo-elastic" phase of the test. The pseudo-elastic portion of the test is where the strains are partially recoverable.

Creep pressure, P_f, is the stress level at the end of the pseudo-elastic zone, before the soil goes into a plastic zone in which deformations are non-recoverable.

The uses of the pressuremeter test results, based upon these parameters, are discussed in Section II of this report. Evaluation of pressuremeter test curves is discussed in Section IV.

2. PRESSUREMETER USES

Engineering Uses

The soil parameters derived from the pressuremeter test (PMT) are based upon typical theoretical considerations of a radially expanding cylindrical surface stressing a homogenous soil. The theory is briefly summarized in Reference 1. The basic formulas resulting from this theoretical analysis have been used to predict soil shear strength, frictional resistance, bearing capacity and lateral earth pressure reactions.

Settlements for structures have been predicted from pressuremeter test results by correlations with a second theoretical analysis; that of a uniformly stressed rigid footing penetrating into a uniform homogenous elastic mass. A brief explanation of this theory is also included in Reference 8.

These two basic theoretical analyses are the basis for all pressuremeter parameter and use definitions. The theory has been correlated in Europe with full scale load tests and laboratory studies, primarily performed by the Center for Geotechnical Studies under the direction of Louis Menard, in Paris. Menard has summarized the uses and the results of the field studies in his report, "Interpretation and Application of Pressuremeter Test Results", 1975 edition. This paper is recommended for all pressuremeter users and is a basic guide to pressuremeter use. It also provides information regarding standardization of test methods.

The typical engineering uses and the appropriate Menard formula are summarized in Table 1. Terms are defined in the List of Symbols, page viii, and in the Glossary, Appendix B.

TABLE 1 - TYPICAL PRESSUREMETER ENGINEERING USES*

ENGINEER'S USE	FORMULA		COMMENTS
Shear Strength	$S_o = \frac{P_l - P_o}{1 + \log_e} \frac{E_d}{2S_o(1+\nu)}$	(1)	Theoretical Derivation P_l P_o and E_d from pressuremeter test curves; ν is Poisson's Ratio
	or $S_o = \frac{P_l - P_o}{2 K_B}$	(2)	Normalized Denominator for ν=0.33; K_B defined for values of E_d/P_l.
Skin Friction	Same as Shear Strength		K_B varies based on the type of foundation & nature of installation, i.e., driven, drilled, etc. value is obtained from charts
Bearing Capacity	$q_{ult} = q_o + K (P_l - P_o)$	(3)	K varies from 0.8 to 9.0 based on depth of embedment, type of soil & shape of foundation, P_l is "Equivalent P_l" above & below bearing level.
Settlement	$W = W_2 + W_3$	(4)	E is equivalent modulus defined for zone being stressed by the foundation; P is load transmitted to the soil; α varies by soil type
	where $W_2 = \frac{1.33}{3E} PR_o (\lambda 2 \frac{R}{R_o})^{\alpha}$	(5)	W_2 is deviatoric settlement
	and $W_3 = \frac{\alpha}{4.5E} P \lambda 3 R$	(6)	W_3 is spherical settlement

* From Menard, "Interpretation and Application of Pressuremeter Test Results" (9)
Terms are defined in the List ofSymbols, p. viii.

Site Specific Uses of the Pressuremeter

Table 1 summarizes the typical uses of the Menard pressuremeter. The parameters described in this table can be used for the various evaluations noted. However, there are specific cases where the pressuremeter can be the most useful method of in-situ evaluation, when compared with other exploration or in-situ techniques. These special situations will be discussed in the following paragraphs.

Testing of Silty Soils

Silty soils are difficult to sample due to disturbance and the lack of cohesion. Carefully retrieved silt soils are sometimes even harder to accurately test, even under the best laboratory conditions. In addition, most theoretical geotechnical analyses are based upon either ideal clays or ideal granular materials and silts fall into a gray zone, which are somewhat ill defined in geotechnical evaluations.

Characteristics of in-situ silts can be quite accurately defined by pressuremeter testing. The tests can be difficult to perform and rely upon very carefully prepared bore holes and careful test procedures. However, when properly performed, they can be very useful in helping to establish in-situ silt characteristics. Case History A, (in Section 6), indicates a typical use of the pressuremeter in a silt soil, where the testing resulted in successful project completion.

Testing In Brittle Soil Formations

Very brittle formations, such as a glacial hardpan or sometimes weathered rock, are also very difficult to sample without causing major disturbance to the sampled material. Similarly, tests upon small portions of the samples that are obtained may not be representative of the actual in-situ conditions. The pressuremeter has been very valuable in helping to evaluate the strength and settlement characteristics of these brittle materials. The bore hole disturbance, typically caused by driving a split-barrel sampler, can be avoided by careful rock-bitting or core-type sampling.

Case History B indicates several projects where the pressuremeter was used to evaluate settlement characteristics of brittle soil formations. Two high-rise structures were to be supported upon drilled pier caisson foundations situated upon glacial hardpan materials. Performance monitoring of these structures indicates that the pressuremeter predicted settlements were very close to the actual settlements observed.

Interbedded Sand, Silt or Clay Soils

Soils with interbedded, non-uniform layers can be difficult to accurately evaluate in the laboratory. For example, a thin silt seam lying within a general clay matrix could result in significant error and problems for triaxial or unconfined strength testing. However, this same silt seam, when tested in the field with the pressuremeter, may result in valid test data relating to the overall soil formation. The large soil volume stressed by the pressuremeter probe seems to even-out inconsistencies in soil deposit layering. This is not to say that the consequences of thin seams or similar potential failure surfaces should be overlooked, but that pressuremeter testing in conjunction with accurate sampling and laboratory testing may provide additional information not obtainable to either alone. Case History C (in Section 6) provides an example where the apparent random layering of a soil deposit significantly influenced laboratory shear strength, but the in-situ testing indicated significantly higher strength values.

Testing of Fill Soils

Due to their heterogenous nature, fill materials can be very difficult to evaluate. This relates to both natural soil fills as well as miscellaneous fill materials.

Pressuremeter testing in fills has been quite useful. Typically, it is necessary to perform a larger than typical number of tests to evaluate the wide range of possible fill conditions, but the results have been quite accurate for a number of different fill materials, including clays and silts and ranging up to very dense well graded granular soils. Case History D indicates a typical use wherein the

shear strength of an old clay fill embankment which was determined by pressuremeter testing was used in the stability analysis of the clay fill embankment. Subsequent performance monitoring indicated that the evaluations based upon the pressuremeter test results were accurate.

Other types of fills can also be tested with the pressuremeter. Case History F summarizes a situation where an old refuse dump was tested with the pressuremeter. The results were remarkably consistent and definable with the pressuremeter tests. At this particular site, the pressuremeter was also used to evaluate the effectiveness of dynamic compaction.

Friction Resistance of Piles or Drilled Piers

Menard has correlated the use of pressuremeter test results with frictional resistance of extended foundations. This is basically through correlations with the soil shear strength and the results have been charted for easy reference. Case History E (in Section 6) indicates the validity of the pressuremeter formula based upon a research project where caisson foundations were loaded to failure. The results correlated well with standard geotechnical methods of evaluation.

Testing in Soft Soils

Results of pressuremeter testing in soft clay soils can be very difficult to obtain with the Menard pressuremeter probe. This is generally due to bore hole disturbance. The most accurate method of forming a bore hole is by augering or careful rock-bitting using drill mud. Case History G indicates a project in which pressuremeter results were evaluated versus in-situ vane shear testing, and large diameter unconfined compressive strength laboratory testing. In this particular case, the shear strength correlations were quite close, which show that, if properly performed, the pressuremeter can provide valid information in very soft deposits. However, the careful preparation of the bore hole cannot be overstressed.

Lateral Soil Parameters

The pressuremeter imparts a lateral stress to the soil or rock formation. In this way, it is measuring the lateral characteristics of a deposit. The nature of this horizontal stressing has been useful in evaluating the horizontal earth pressure characteristics with regard to overturning of transmission towers, as indicated in Case History H. The lateral parameters defined in the pressuremeter tests were much better than the engineered guess based upon standard penetration blow count values.

Case History I also indicates a situation where the pressuremeter was used to predict movements of a pile foundation subjected to lateral load. The lateral resistance of granular surface fill materials which was necessary in providing the required overturning resistance was accurately predicted by pressuremeter testing.

The proceeding paragraphs indicate some of the prime uses for pressuremeter testing. In each case, the pressuremeter was used to supplement or supersede standard methods of sampling or analysis. In cases where performance monitoring was available it appears that the pressuremeter test results, combined with sound engineering judgement, provided accurate and predictable results.

3. PRESSUREMETER USE LIMITATIONS

The Menard Pressuremeter test can be affected by many different conditions of the test itself. Some have a significant influence on the results, whereas, others have only a slight influence. The following paragraphs briefly summarize conditions which can affect pressuremeter test results.

Variations in Pressure Increments

The pressure increments or resulting number of test points can have a significant affect on test results. The smaller the pressure increment and consequently, the larger the number of points for a test curve, the "steeper" a curve will be. This results in a lower value of modulus, E_d, and a lower value of limit pressure, P_l. Menard has attempted to standardize the test curve by specifying that it should consist of between 7 and 15 separate points. Schmertman has indicated that the pressure increments can significantly affect the drained/undrained condition of the soil and that the resulting pressuremeter curve may indicate that the first portion of the curve is essentially drained and the latter portion is essentially undrained. The Soil Testing Services, Inc. experience has been that nearly all soil types are affected by changes in pressure increments, with the exception of perhaps the hardest high strength hardpan or soft rock formations. References which comment on pressure increment changes include Roy, et al, Reference 12.

Temperature of Test Liquid

Roy, et al, (reference 12) notes that the temperature of the pressuremeter test liquid tends to influence the test curve. The lower the temperature, the "flatter" the curve becomes, and hence, the higher the values of E_d and P_l. The effects are relatively small and would only require correction for very soft clay or low limit pressure soils. He also notes that the viscosity of the test fluid may slightly alter the pressuremeter test results.

Bore Hole Versus Probe Size Ratio

The size of the bore hole into which the pressuremeter is inserted must have a "snug" fit to ensure optimum PMT results. A size relationship has been determined by Roy, et al (reference 12)as follows:

$$Q = \frac{\text{diameter of the bore hole}}{\text{diameter of the probe}}$$

As 'Q' increases the bore hole size is larger than the probe and the value of modulus, E_d decreases. The literature (references 6, 12) has cited a reduction in E_d of between 20% and 40%, or more as 'Q' increases. Thus, the effect on the modulus, E_d, is significant. It is generally felt that when the pseudo-elastic portion of the curve is reached, the probe volume should not exceed 220 cc, for valid test results. This constraint limits the value of 'Q' to 1.10 for a BX size Menard probe. The Soil Testing Services' experience has been similar to the cases cited in the literature, that is, Q has a significant effect upon E.

However, the value of Q does not apparently affect the limit pressure, P_l, to any great degree.

Probe Size Changes

Roy, et al, has also cited the effect of probe size on pressuremeter parameters. For the relatively soft clays tested, the limit pressure, P_l, appeared to be proportional to the size of the probe, at least for the probe sizes changing from AX to BX size. In this case, changing probe size was also a change in the probe length to diameter ratio. Laier, Schmertmann and Schaub, (reference 7) have found that the length to diameter ratio has a marked effect on measured limit pressures. As the length to diameter ratio increased, the limit pressure decreased. The reason for this is the larger area is stressed in a "spherical" expansion rather than a "radial" expansion, when a larger probe length to diameter ratio is used. Laier has formulated correction factor curves to convert test P_l values to P_l values for a theoretically infinite probe length.

The Soil Testing Services' experience has been predominately with a single size probe, the BX size probe. Initially, NX and BX size probes were utilized. However, it was found easier to form a hole for the BX probe and the use of the NX probe was dropped several years ago before any correlations between the two were attempted.

Tests in Soft Clay

In soft clay deposits, the soil undergoes a slow creep during the time required to perform a field test. This creep is one reason for the limitation of pressure increments as noted above. Roy, et al, designed a test wherein the pressuremeter was pushed directly into a soft clay formation and a certain time period was allowed before the test was performed. The test results obtained after the "relaxation" period was allowed were significantly lower than when the soil was tested immediately. Pore pressure dissipation was cited as the reason for the changes noted. This is an unusual procedure, and direct comparison or correlations with typical Menard test procedures can not be made. However, it does imply that the time in which a test performed and the state of the soil being tested, i.e., drained, undrained, etc., can have a significant affect upon very soft soil conditions.

The difficulty of testing in soft clays has been observed by many authors and is the prime reason for development of the self boring pressuremeters. The literature details problems in forming hole size, squeezing of soft soils, etc. (references 1, 2, 8, 9, 10, 12, 13).

There is also a problem of pressure control when using a conventional Menard pressuremeter in soft clay at deep levels. As soon as the valve is opened to the inner membrane, the pressure due to the water in the line may be sufficiently high to be beyond the creep pressure so the initial portion of the test curve is "lost". Special control valves should be inserted immediately above the test probe to allow for application of small pressure increments.

4. EVALUATION OF PMT CURVES

Sampling Technique

The limitations to PMT results based upon specific test conditions, such as bore hole size, method of sampling and the shape of the pressuremeter probe are important. However, the interpretation of the resultant PMT curve is equally important. The relative differences between the theoretical considerations regarding laboratory performed correlations and actual data obtained from in-situ test conditions should not be overlooked. Field Pressuremeter test curves and resulting parameters can vary substantially based upon the soil and rock conditions encountered. This section deals with the evaluation of PMT curves.

General Evaluation of PMT Curves

Actual PMT data frequently plots in the form of the ideal curves shown in Figure 2. However, in many instances, the pecularities of the soil formation or of the specific test procedure result in PMT curves which may be significantly different and yet will provide substantial information regarding the soil or rock deposit. Since the Menard Pressuremeter can be used in a wide variety of soil and rock formations, from relatively soft or loose to extremely dense or soft rock formations, the intrepretation of PMT results varies, depending upon the specific condition. The Menard Pressuremeter rules have accounted for some of the differences in PMT curve formation.

In very soft soil formations, the bore hole disturbance resulting from use in the Menard Pressuremeter probe typically results in a poor value of modulus E_d. However, it appears that the limit pressure obtained from a test in soft materials is not significantly altered. In general, the limit pressure values, P_l, in soft materials are more accurate than the modulus values E_d.

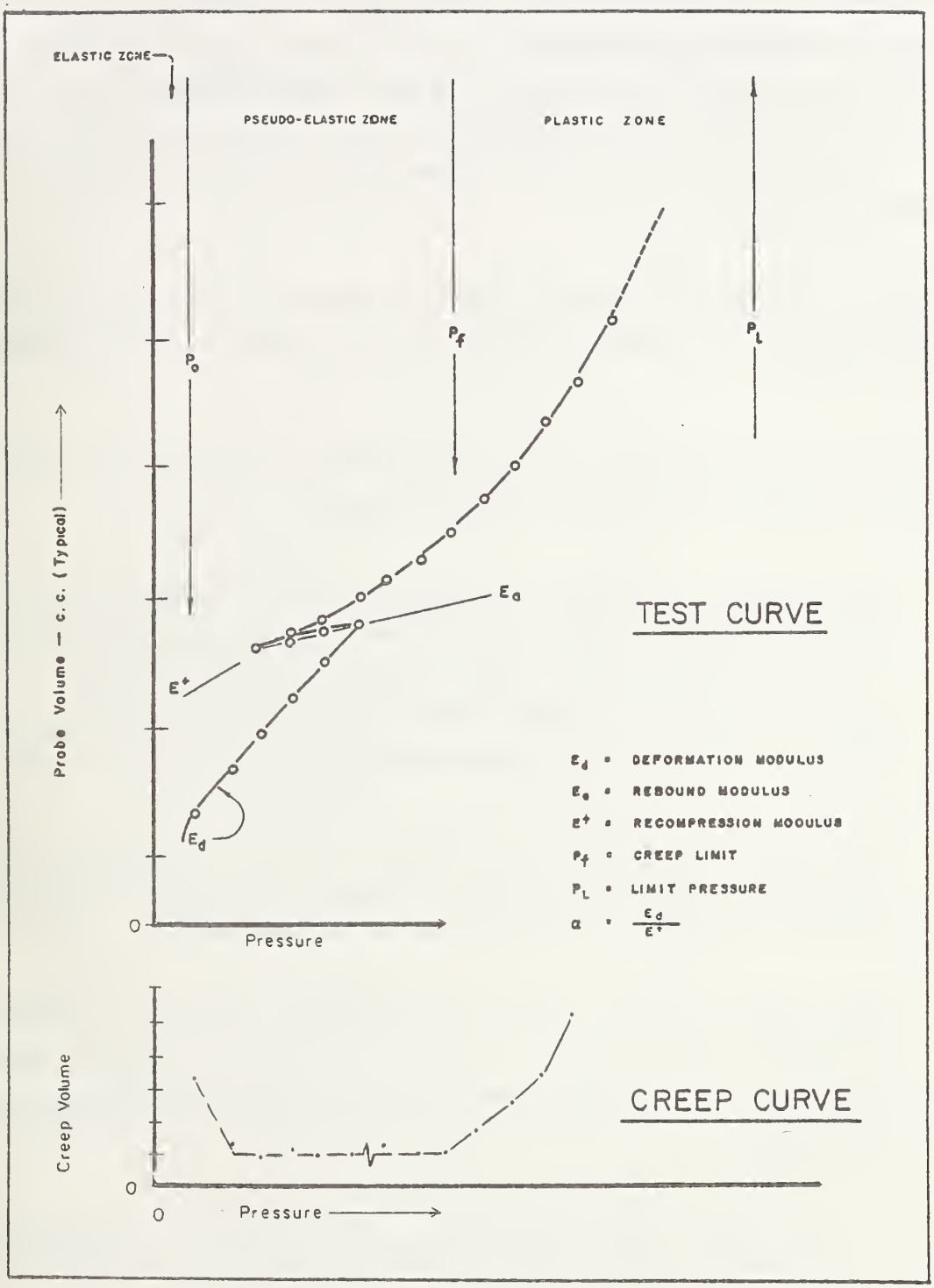


FIGURE 2 - PRESSUREMETER TEST AND CREEP CURVES

Conversely, in very dense soil or soft rock formations, the PMT curve results in a very well defined value of modulus, E_d, but the typical pressures compatible with most pressuremeters will not allow for a high enough stressing of the soil to provide good values of limit pressure, P_l. Thus, in these types of dense or soft rock formations, the most accurate value determined from the PMT is the modulus value, E_d.

Thus, it can be readily seen that the nature of the soil or rock formation and the test results can have a significant effect upon the accuracy of the data obtained from the PMT.

To help evaluate the accuracy of the specific PMT curves, four general shapes have been defined as follows and as shown in Figure 3.

Curve-Type A - is defined as having a minimum of three points on the straight line, pseudo-elastic portion of the curve, having the initial portion of the pseudo-elastic curve begin at a value of Q less than 1.10 (220 cm of expansion for a BX size probe), and the last point on the curve is 75% or greater of the limit pressure determined for the particular test.

Curve - Type B - consists of at least three points on a straight line in the pseudo-elastic portion of the curve, a Q value of one less that 1.10 and the last point on the curve greater than 60% of the limit pressure.

Curve Type C - consists of at least five points on a straight line defining the elastic modulus, E_d, a Q value of less than 1.10, but not enough information to accurately determine the limit pressure from a geometric construction.

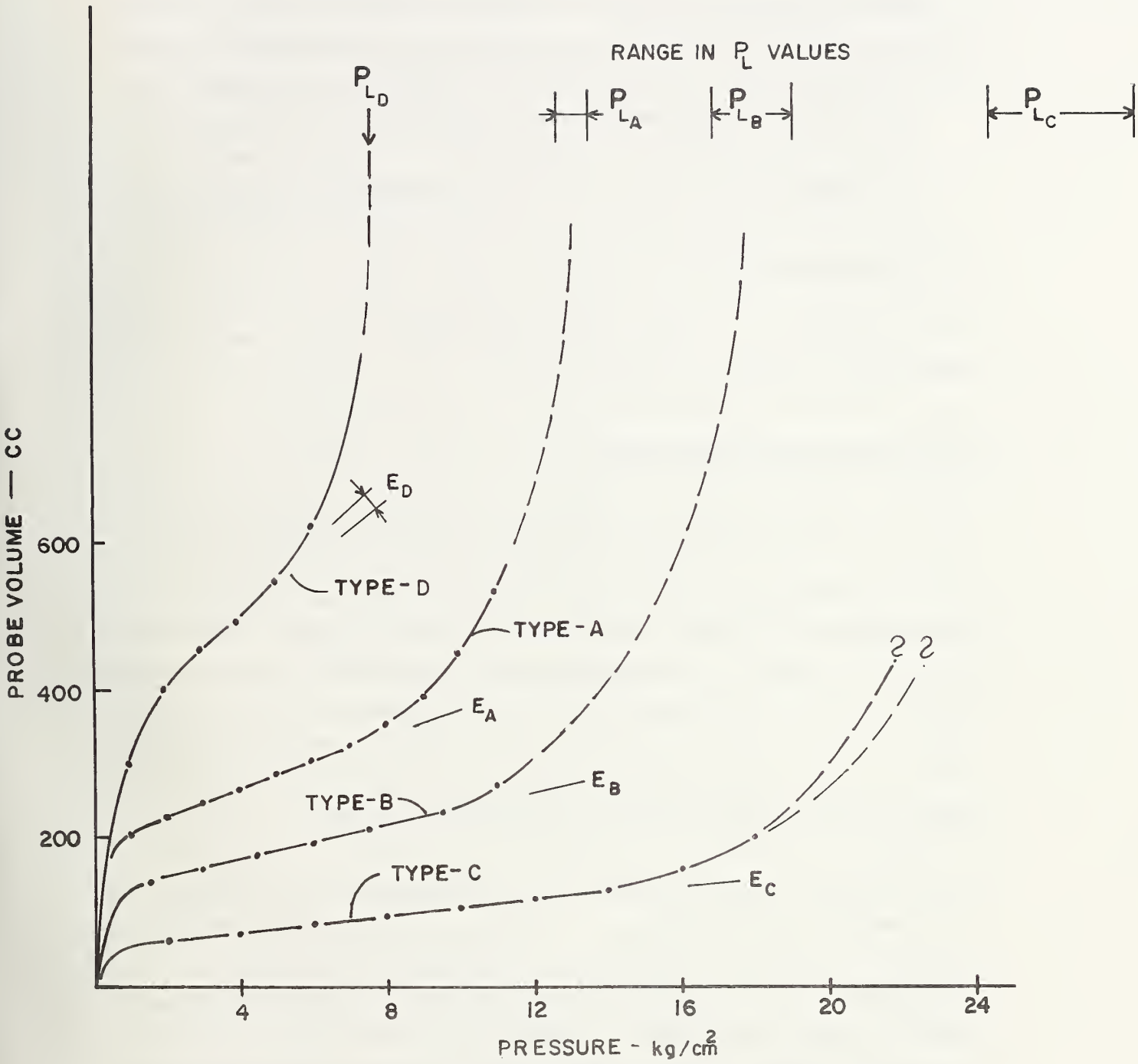


FIGURE 3 TYPES OF PRESSUREMETER CURVES

Curve - Type D - consists of other curves which appear to provide information that may not fall into the above mentioned categories. Examples of valid curves may be those obtained in a soft deposit where only two points may be defined in a straight line portion for the modulus E_d and yet the limit pressure may be valid, or a situation where the test results indicate a progressive failure during testing and therefore, an odd shaped PMT curve, yet still one indicating valid information regarding in-situ conditions.

The ideal PMT results would fall into Category A. However, in-situ, many results will fall into other categories.

Determination of Limit Pressure P_l

Menard has defined the determination of limit pressure as the asymtotic shape of the PMT curve where the volume of the probe increases very rapidly (to infinity) under very small increases in pressure. Carrying the PMT to completion under every situation would result in a burst membrane for every test. Since this is not generally feasible, the tests are carried to a predetermined volume and then ended. For uniformity, Menard defines the limit pressure as a value where the probe expands to twice its normal size. When plotted on a semi-log curve, this results in a straight line and the limit pressure at a value of 700 cc (for a BX size probe) it has been arbitarily defined as the limit pressure value.

The STS experience has been that, in some cases, this arbitrary definition of limit pressure, P_l, is appropriate. However, in cases where the PMT is performed in a brittle or very soft soil, this generalized definition of P_l may be inappropriate. It is our opinion that the most accurate definition of P_l can be obtained from an extrapolation of the shape of the PMT curve where the specifics of the soil conditions allow for this extension, noteably Type A and Type B curves. Where a curve falls into the Type C or perhaps Type D category, the Menard generalization is the best method of determining the P_l value.

For Type A and B curves, the difference between Menard's construction and graphical extrapolation of test data was $\pm$ 5% for 25 typical test curves evaluated. For a similar 25 test curves of the Type C and D category, the range was up to $\pm$ 25% of the Menard construction, for the graphic P_l value.

Determination of Modulus Value E_d

The pressuremeter modulus value, E_d, is the straight line portion of the PMT curve. For all except the softest soil conditions, a valid test will usually have three or four points on a straight line portion, which makes determination of the modulus value relatively easy. In general, it is the most accurately determined parameter from the test curves. In a soft soil deposit, or a deposit which is undergoing progressive failures, the curve shape may be non-linear and the determination of E_d is somewhat more difficult. This is usually due to bore hole disturbance, sampling disturbance or non-uniform soils being tested.

Since the modulus value is a volumetric strain modulus based upon the size of the bore hole, it is important that it be determined only along the straight line portion of the PMT curve. The average volume required in Menard's determination of modulus should be taken as the volume at the mid-point of the straight line portion.

In some soil conditions, the rebound modulus, E_r, may be determined by unloading the soil sample a portion of the way through a test, and then reloading the sample to the end of the test. The rebound modulus, E_r, is the straight line of slope of the reload portion of the test.

Several years ago, the ratio between the rebound modulus and the regular modulus was thought to be a way to define the coefficient alpha, α. However, correlations do not appear to be consistent and the use of this method to determine alpha has generally been dropped by most pressuremeter users.

Determination of Creep Pressure P_f

The creep pressure is typically defined as the point at which the soil changes from the pseudo-elastic phase to the plastic phase. On a pressuremeter curve, it is typically at or just beyond the end of the straight line portion of the curve. The best determination of creep pressure can be made by graphing the values of creep on a separate axis, i.e., creep versus pressure. The plot of the creep curve typically has two or more straight line portions. The break point from the flattest portion of the curve to the next zone is typically taken as the creep pressure, P_f.

In a moderately strong, cohesive soil or a medium to dense granular soil, the creep curve is typically well defined and the corresponding creep pressure can be quite accurately determined. In a very dense formation, the transition in creep curve is sometimes quite gradual, as is the transition of the pseudo-elastic to plastic zone and the corresponding creep pressure is less accurately defined. In a soft formation, the creep curve may or may not be well defined and the corresponding creep pressure determination may be less accurate in these conditions.

Pressuremeter Correlation Ratios

Two ratios determined from the test parameters are very useful in proper evaluation of the parameters. They are the E/P_1 ratio and the P_f/P_1 ratio.

The E/P_1 ratio is somewhat characteristic of the soil type. The softer or looser the deposit the lower the value of E/P_1, typically in the range of 5 to 8. For normally consolidated soils, the range is on the order of 8 to 12. For overconsolidated soils, the value is between 12 and 20, or higher. A graph of E/P_1 versus log E shows characteristic trends for various types of soil. This will be further discussed in the in-house research section of this report.

The P_f/P_l ratio is also useful in evaluating the accuracy of PMT results. This ratio will typically be on the order of about 0.55. The range is usually between about 0.4 to 0.7. In general, soft or loose soils have a lower value of P_f/P_l ratio and more brittle or dense soils, or overconsolidated soils may have a higher ratio.

The use of the E/P_l and P_f/P_l ratios are useful in evaluating Type C curves. This is where the modulus value E is well defined and the creep pressure P_f is well defined but the curve may not have been extended far enough to allow for an accurate determination of P_l. Knowing the type of soil and its approximate overconsolidation ratio, a generally expected P_f/P_l ratio can be assumed. The approximate P_l value is obtained by dividing P_f by the P_f/P_l ratio. A check on the approximate E/P_l ratio and correlation with Menard's graphic construction of P_l will help evaluate the validity of the P_l value selected.

Summary

The steps in the most effective determination of pressuremeter parameters from test curves may be summarized as follows:

1. Graph the volume versus pressure curves and volume versus creep curves (for corrected pressure values).

2. Determine preliminary values of E_d, P_l and P_f. Define the general shape of the curve as A, B, C or D as previously discussed.

3. Calculate the E/P_l and P_f/P_l ratios.

4. Starting from the most accurately determined value for each test, (e.g., E_d values for Type C curves, or P_l values for Type D curves), re-evaluate the other parameters in conjunction with the E/P_l and P_f/P_l ratios.

5. If, after calculating or redetermining the values of E, P_l and P_f and correlating with E/P_l and P_f and P_l ratios, the parameters and ratios fall into typical ranges, the test results can be assumed to be representative of the deposit tested.

5. IN-HOUSE RESEARCH

Through our use of the pressuremeter since 1970, Soil Testing Services, Inc. has accumulated a large amount of data relating to actual pressuremeter testing in all types of soil and soft rock. Wherever possible, an attempt has been made to correlate the pressuremeter parameters with standard laboratory or field determined soil parameters. We have also tried to further define pressuremeter parameters in relation to soil types. As a third general area of in-house research, we have worked toward establishing the optimum field conditions for pressuremeter testing and are developing a bore hole formation instrument to help to obtain more accurate results in loose or soft soils.

Correlations With Standard Soil and Rock Parameters

Correlations with soil shear strength are made wherever possible in cohesive soil deposits. This is done by comparing the laboratory determined unconfined compressive strength with that defined by the Menard Formula, which is as follows:

$$S_o = \frac{P_l - P_o}{2 K_b} \qquad (2)$$

The term 'K_b' has been defined by Menard for different values of E/P_l. For a cohesive soil within E/P_l equal to 10, K_b has been defined as 2.75. Our experience has shown that K_b can range from between 2.4 to 2.8, with an average value being 2.55. This has been previously recorded in a paper by STS personnel Lukas and deBussy (reference 8, September, 1976) and has since then been correlated with numerous other test results.

Correlations with consolidation test parameters have also been demonstrated by Lukas and deBussy. A correlation between the consolidation test constrained reload modulus D_c, has been defined in relationship to the pressuremeter modulus, E_d as follows:

$$4.5\ E_d \leq D_c \leq 6.0\ E_d \tag{7}$$

The relationship just defined shows significant data scatter, but appears to provide the approximate order of magnitude for the relationship. Further correlations and other geographic areas are being studied to help to define the relationship more accurately.

Also, as related to cohesive soil, the relationship between creep pressure, P_f and the preconsolidation of pressure, P_c, has been evaluated. Several references (reference 8 and 11) have defined the approximate relationship as follows:

$$P_f = P_c \tag{8}$$

The Soil Testing Services' experience has been to confirm the general relationship just given.

Correlation with Standard Penetration Resistance Values "N", have been attempted. We have been unable to define any accurate relationship between limit pressure, P_l, versus 'N' value or pressuremeter modulus, E_d, versus 'N'. In general, as the standard penetration resistance value increases, the limit pressure and modulus values increase. It is well recorded that the relative density, gradation of granular soils, and depth influence the standard penetration value. However, the same constraints apparently do not effect the pressuremeter values in a way which allow for accurate correlations. This may be due to the fact that the pressuremeter applies relatively slow stresses in a horizontal direction and the standard penetration split-barrel sampler impacts in a vertical direction. In addition, the standard penetration resistance values have been shown by investigators (references 1, 9) to vary considerably, depending upon operational techniques and field conditions.

Even in relatively uniform soils, we have found no relationship accurate enough to allow an engineer to correlate 'N' values to pressuremeter parameters. For example, Figure 4a indicates standard penetration resistance values versus limit pressure values for a uniformly graded, micaceous silt from the Washington, D.C. area. No specific correlations can be observed.

Correlations Between Pressuremeter Parameters

The relative accuracy of the various determinations for pressuremeter parameters has been discussed in preceeding sections of this report. Since values of E_d and P_l are used throughout the basic Menard Formulas, it is critical that they be properly evaluated from the PMT curves. We have found that certain relationships appear to be definable for various soil types. These relationships can be useful in definition of the most accurate test results. The basis for the following correlations is approximately one hundred projects and over five hundred pressuremeter tests.

A useful relationship can be defined between E/P_l versus $\log E_d$. This relationship appears to be linear, as shown in Figure 4b for the same Washington, D.C. micaceous silt just described.

The slope of the linear relationship and the position along the $\log E_d$ scale appears to change with soil type and previous soil/stress history. For example, Figure 5 indicates curves for three different kinds of silt. In addition, Figure 6 has been drawn for normally consolidated fill materials. The slope is much steeper and the position somewhat to the left of the curves shown in Figure 4.

Figure 7 shows on one graph the various relationships which have been observed.

There are several uses for the curves just described. For example, if a soil type is well known and its apparent horizontal stress history can be anticipated, the appropriate limit pressure value, P_l, may be accurately defined for a Type C curve by establishing the relationship between 'E' and 'E/P_l' from a typical soil type curve. By entering the curve with the modulus value, the corresponding E/P_l value can be determined and consequently, the P_l value.

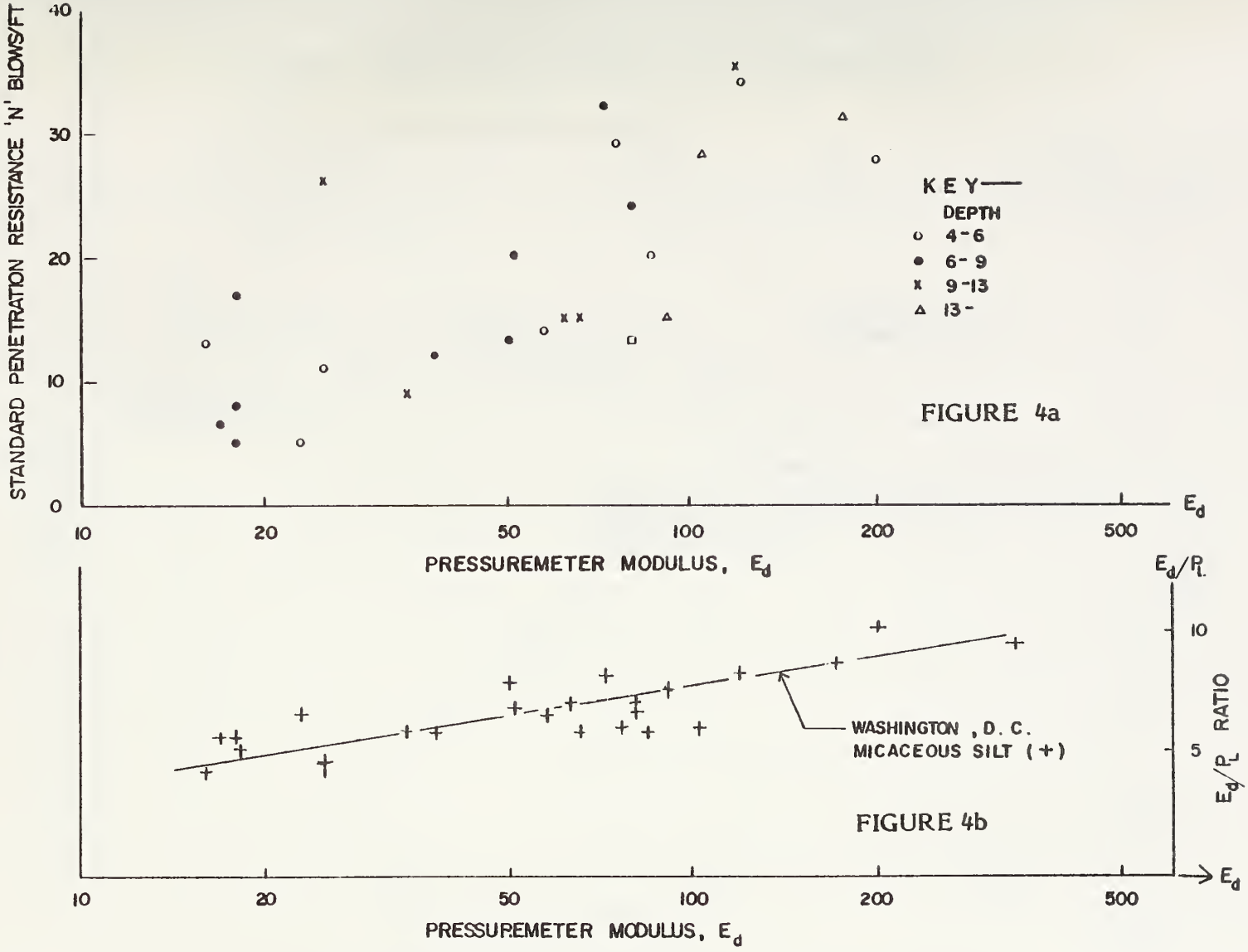


FIGURE 4 - SOME PMT RELATIONSHIPS FOR A MICACEOUS SILT

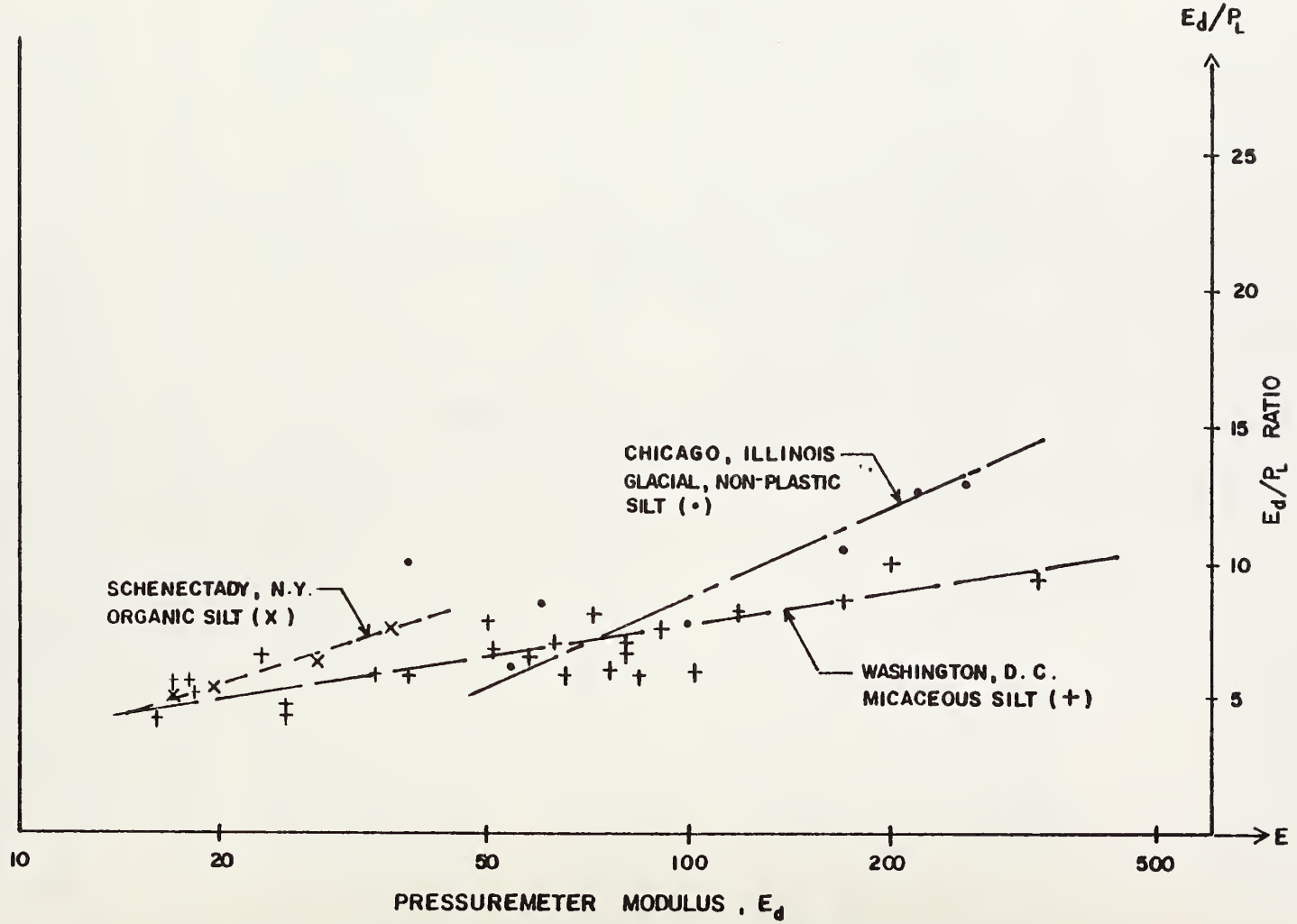


FIGURE 5 - E_d/P_l RATIO VERSUS E_d FOR THREE SILTS

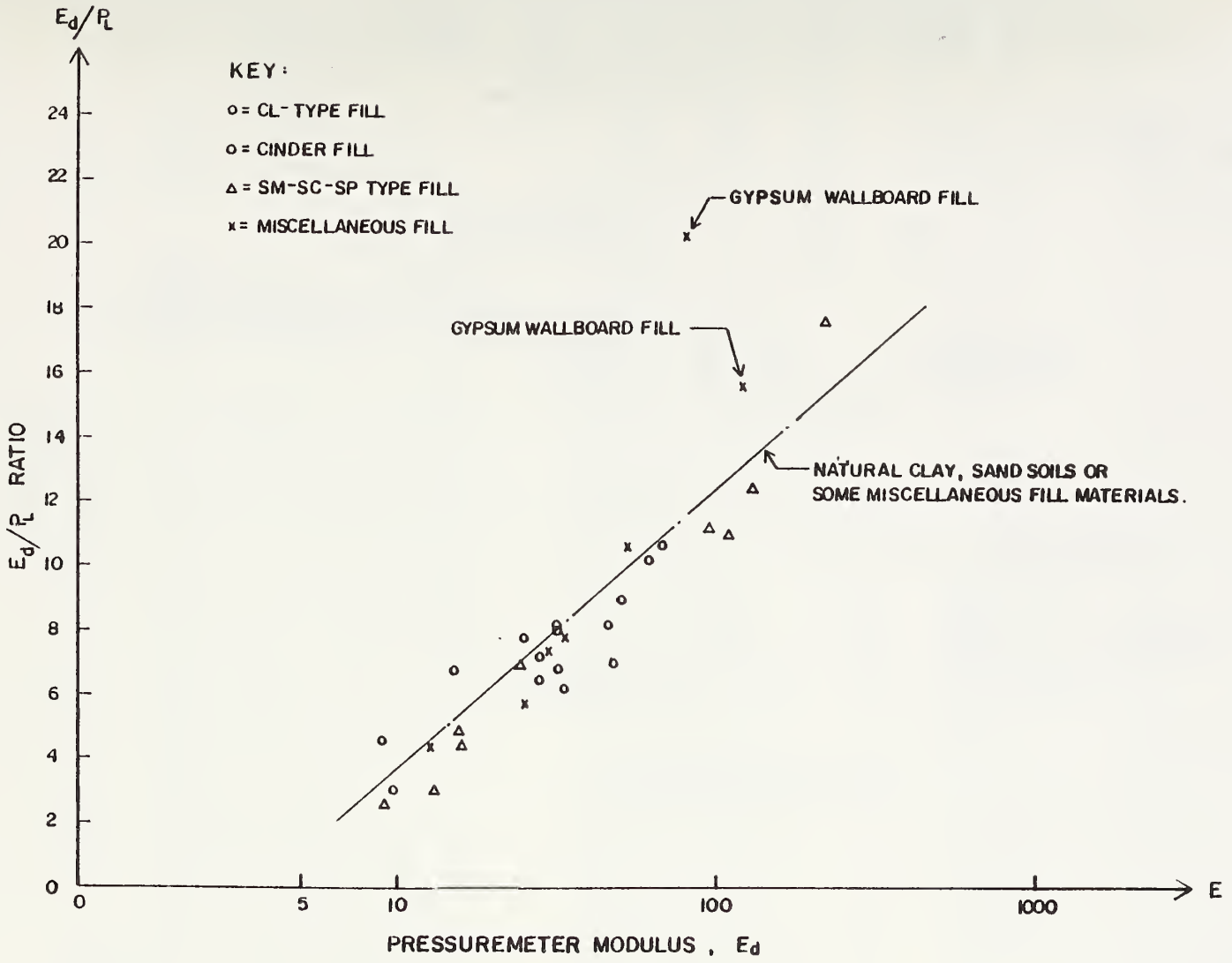


FIGURE 6 - E_d/P_l RATIO VERSUS E_d FOR FILL MATERIALS

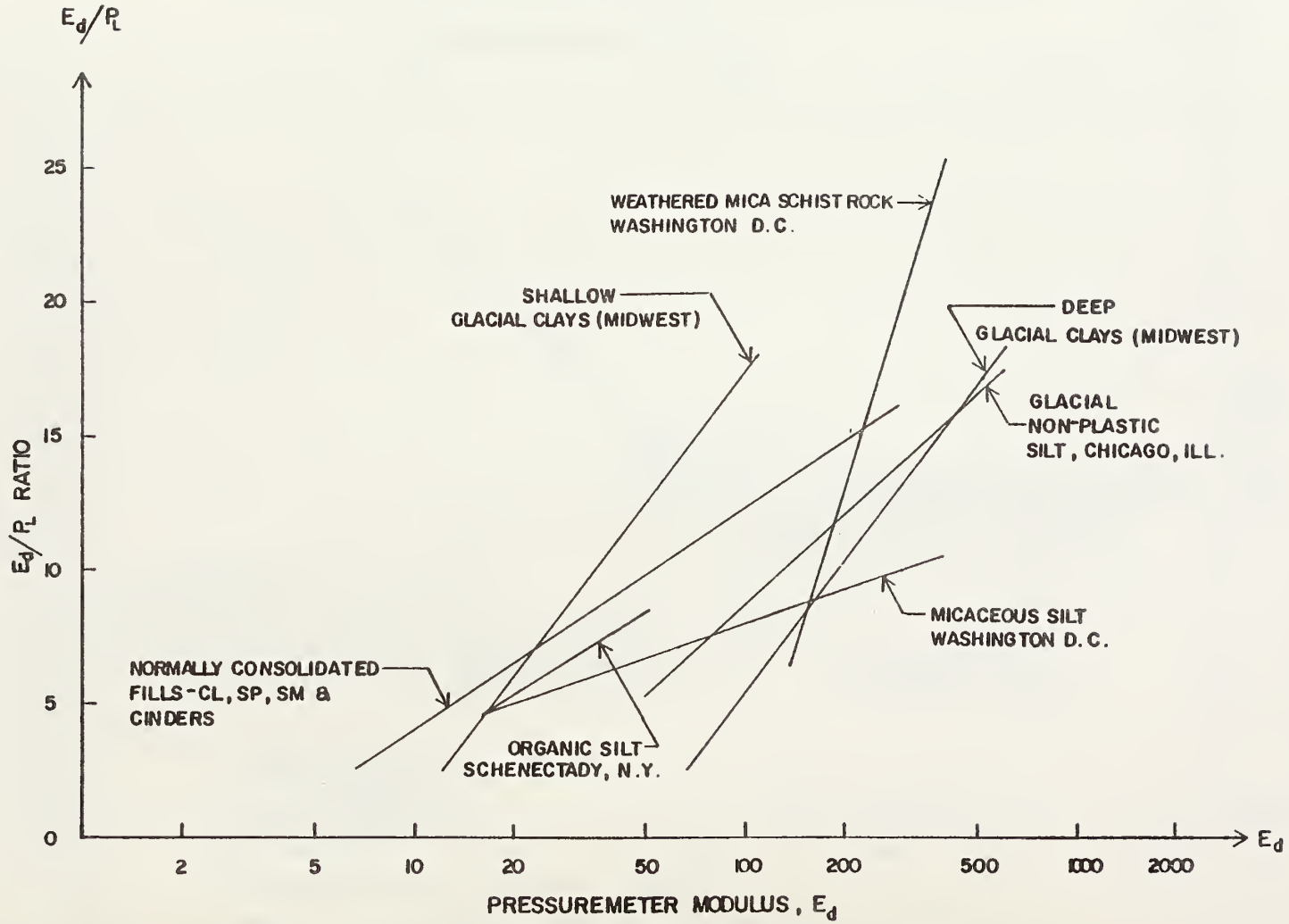


FIGURE 7 - E_d/P_l RATIO VERSUS E_d FOR VARIOUS SOILS

Conversely, the approximate stress history of an unknown soil may be approximated from a Type A or Type B pressuremeter curve for cohesive soils. By knowning an accurate E/P_l ratio and modulus value, the samples' relative position between the "normally consolidated" curve and the "highly overconsolidated" curve can be defined and the approximate stress history estimated. As further data is obtained, families of curves may be definable, resulting in more accurate predictions.

Similar relationships have been attempted for E/P_l versus log P_l and P_f/P_l versus log E_d. Similar relationships, but not as accurately defined as those for E/P_l versus $\log_e$ are apparent.

Bore Hole Formation

The need for the most accurate bore hole formation was recognized early during the Soil Testing Services' use of the Menard Pressuremeter. We have continuously tried to correlate the test result accuracy with bore hole formation. Evaluations have included the Menard slotted casing, hole formation by samplers, either thin wall tube samplers or split-barrel samplers and bore hole formation with special cutting tools.

We are in the process of developing a special sampler called the "Lukas Shaver". In this method of operation, a rock bit using drill mud, which is just smaller in diameter than the desired probe size, is used to drill the probe hole. This smaller diameter rock bit is followed on the drill stem by a thin wall shaver, which does not rotate as the bit is extended in depth. The shaver is exactly the size of the probe hole desired. The process allows for the drill mud and cuttings to flow up between the thin wall shaver and thereby, minimize the bore hole disturbance. There is no suction as the tool is withdrawn and the drill mud keeps the bore hole open.

Another development under study is a modification of the valving arrangement of the water line to the inner membrane to allow small pressure increments to be applied to the soil. This would be important when testing soft soils at great depths since the pressure due to the height of water on the inner probe can be significant at the greater depths. Oftentimes, this pressure is greater than the creep pressure, so the initial portion of the pressuremeter curve is obliterated.

The following table summarizes our best estimation of bore hole formation for all types of materials for which we have tested.

TABLE 2 - TECHNIQUES FOR BORE HOLE FORMATION FOR PRESSUREMETER TESTING

SOIL OR ROCK DEPOSIT	MOST SUCCESSFUL METHOD OF PROBE-HOLE FORMATION FOR BX PROBE
Soft to Medium Firm Clay (Qu $\leq$ 1.0 to 1.5 tsf) a) Shallow, $\leq$ 25'	Sample with 2" diameter thin-wall tube pushed 2.7' into test stratum; shave sides with 2.5" thin-wall tube pushed to within 0.2' of bottom of 2" diameter pilot hole - shavings will fall into bottom 0.2' and suction upon retrieval will be minimized; use drilling mud below 5'
b) Deep, $\geq$ 25'	Use drilling mud; rock bitting with a 2.37" diameter bit may be better than above thin-wall tube technique in squeezing formations
Stiff to Very Stiff Clay (Qu $\leq$ 4 to 5 tsf)	Form pilot hole with 2.0" diameter thin-wall sampler, pushed 2.0'; form probe hole with a 2.5" diameter thin-wall tube, pushed 2.5'
Hard to Very Hard Clay (Qu $\geq$ 5 tsf) a) Not Gravelly	Try same technique as for very stiff clay; if penetration of tubes is not sufficient, use a 2.37" diameter rock bit for hole formation.
b) Gravelly	Use 2.0" diameter split-barrel sampler to form pilot hole; follow with 2.5" diameter split-barrel sampler for probe hole.

TABLE 2 - TECHNIQUES FOR BORE HOLE FORMATION FOR PRESSUREMETER TESTING (continued)

SOIL OR ROCK DEPOSIT	MOST SUCCESSFUL METHOD OF PROBE-HOLE FORMATION FOR BX PROBE
Silts	
a) Dry, dense	Use 2.5" diameter split-barrel sampler.
b) Moist, loose	Very difficult, usually experimentation is required to define best technique; use drilling mud, try 2.0" followed by 2.5" split-barrel sampler or try 2.37" diameter rock bit; 2.0" diameter auger may be successfull at shallow depths, less than 10'
Sands	
a) Loose, non-water bearing	Use 2.5" diameter split-barrel sampler driven 2.5' (no pilot hole).
b) Loose, saturated	Use normal to thick consistency drilling mud; drive 2.5" diameter split-barrel sampler or use 2.37" diameter rock bit slowly advandanced in low gear.
c) Dense, non-water bearing	Form pilot hole with 2" diameter split-barrel sampler; follow with 2.5" diameter split-barrel sampler.
d) Dense, saturated	Same, but use drilling mud; may require 2.37" diameter rock bit.
Gravel	
a) Loose or open graded, moist or saturated	Very difficult due to caving; experimentation is usually required to define best technique; possibly try slotted casing.
b) Dense, well graded, moist or saturated	Same as for dense sands.
"Soft" Rock	Use drilling water and 2.5" diameter rock bit at a slow enough speed to minimize stratum disturbance.

VI. CASE HISTORIES

Case Histories are an excellent method to demonstrate typical pressuremeter applications and methods of evaluation. The following case histories have been selected to be representative of the prime uses of the pressuremeter as discussed in Section II. The nine case histories which follow present engineering evaluations based upon pressuremeter tests, including settlement calculations, bearing capacity, soil shear strength, foundation side friction, slope stability and lateral resistance for piers and piles. Soil conditions ranging from soft clays to extremely dense hardpan materials have been discussed in these case histories.

A. SHALLOW FOUNDATIONS OVER SOFT SOILS - FOOTINGS ON SILT

General

A building in the vicinity of Schenectady, New York was used for the construction of large steam turbines. There are a number of cranes that operate in the building, and the largest operating crane can lift a 100 ton (90,700 kg) load. Larger turbines were to be built and it was desired to lift a total of 200 tons (181,400 kg) by the addition of another crane so that both cranes could work in tandem. Previous soil explorations had indicated very soft silt soil conditions. The pressuremeter was suggested as an in-situ exploration tool to help evaluate the silty soils.

Project Information

Spread footing foundations support the structure which was constructed in 1903. They are situated on a 10 to 15 foot (3 to 5 m) thickness of loose clayey silt that is encountered below the footing bearing level. Miscellaneous granular fill and cinders are present above footing depth (7.5 ft., 2.1 m.) to the floor slab level.

In general, the clayey silt on which the foundations are bearing had a unit dry weight of 95 pcf (1.52 kg/cm^3), water contents between 30% and 35%, unconfined compressive strengths of 0.5 tsf (0.49 kg/sq.cm) and standard penetration blow counts of 0 to 5. The foundation loads transferred to the soil bearing stratum from the exterior and interior columns were on the order of 800 and 1500 psf (0.39 kg/sq.cm and 0.73 kg/sq. cm) respectively, prior to the additional crane loads. These loads exclude short-term crane loading. With the additional crane utilized to lift the heavier loads, the total foundation pressure would be on the order of 2800 psf (1.37 kg/sq.cm), two to three times the present loading.

At the time of the pressuremeter testing, an inspection was conducted of the exterior which indicated that the building had performed adequately with regard to differential movements. No major cracks were noted in the brick walls and only a very few hairline cracks could be seen anywhere within the structure.

Use of the Pressuremeter

Even with very careful sampling and handling procedures, silty soils can become disturbed upon removal from the ground. The pressuremeter, because of its "in-situ" test procedure, was used to evaluate more representative, undisturbed conditions for use in bearing capacity and settlement analyses. A series of five tests was performed at two interior locations within several feet, horizontally of the spread footings. Test depths were selected to evaluate soil conditions in the depth zone equal to about two times the minimum footing dimension. Tests were concentrated in the lowest density soils within this depth range.

Results and Conclusions

Conventional settlement and bearing capacity analysis based upon the standard penetration "N" values would have predicted substantial settlements and unstable bearing capacities. Analysis based upon test data from undisturbed, thin-walled samples indicated a safer condition.

For the increased loading, up to 2800 psf (1.37 kg/sq.cm) conventional bearing capacity analysis predicted a safety factor of 1.3 with regard to failure. Based upon a more representative profile defined by the pressuremeter data, a safety factor was calculated at 2.0. The pressuremeter data is summarized in the following table:

TABLE 3 - PRESSUREMETER DATA SUMMARY

BORING	DEPTH FT.	E TSF	P_l TSF	P_f TSF	P_f/P_l	E/P_l
B-2	11'	17	1.8	0.3	.17	9.5
	15'	36	3.1	1.8	.59	11.6
	20'	42	5.1	1.8	.36	8.2
	25'	42	4.3	1.8	.42	9.8
	30'	35	4.3	1.7	.40	8.2
B-24	10'	19	2.2	0.4	.18	8.7
	14'	28	3.0	0.7	.23	9.3
	18'	43	4.2	1.4	.33	10.2
	24'	62	6.5	2.5	.38	9.5
	29'	66	9.0	4.0	.44	7.3

Note: 1 TSF = 0.96 kg/cm^2

The pressuremeter limit pressure values along with a bearing factor are utilized in the bearing capacity analysis. The bearing factor is dependent upon the soil type, depth of embedment and the size of the footing and is obtained from Menard's Chart. For an average soil profile the bearing capacity calculations are shown belows:

$$q_{ult} = q_o + K (P_l - P_o) \qquad \text{(A-1)}$$

where: q_o = overburden = γh

P_o = $K_o \gamma h$, where $K_o = 1$

K = 1.20 from Menard Chart for Soil 1 category,

P_l = equivalent P_l, or geometric mean of P_l values, which equals 2.46 tsf

q_{ult} = 0.48 tsf +1.20 (2.46 tsf-0.48 tsf)

q_{ult} = 2.86 tsf

Due to the massive footing size, the weight of the concrete should be accounted for in the net bearing value. This results in a net allowable bearing value of 2.73 tsf.

The conclusions were that the small increase in sustained dead loads from the new cranes would not cause any substantial change in building performance with regard to the bearing capacity failure or to building settlement. A bearing capacity factor of safety of 2.0 as computed from the ultimate bearing capacity predicted for the average condition by the pressuremeter at the anticipated maximum load increase. The computations based upon the pressuremeter test results indicated that only a minor increase in foundation settlement would occur from the cranes lifting the additional load of the newer, heavier turbines. If the heavier loads were left in-place for a long period of time, a substantial differential settlement was predicted and possibly large settlements associated with shear failures of the soil. Minor cracking of walls was then predicted. Fortunately, the new turbine loads would be lifted by the cranes only six times a year for relatively short periods of time.

B. DEEP FOUNDATIONS SUPPORTED UPON PRECONSOLIDATED COHESIVE SOILS

General

In glacial areas, the glacial till deposits are frequently highly preconsolidated and provide a good foundation bearing stratum for heavy loads. The classification and consistency of the glacial till varies significantly, especially on a regional basis, but frequently, the tills are a mixture of cohesive and granular soils in a very dense state of packing. The deposits are difficult to sample by conventional techniques because the Standard Penetration Resistance Values are frequently in excess of 60 blows per foot and it is difficult, if not impossible, to advance undisturbed sampling devices into this type of formation. When representative samples are obtained, sample preparation is quite difficult because of the presence of gravel particles within the sample and the brittle nature of the material. Design of foundations bearing upon these deposits is frequently based upon the Standard Penetration Resistance Value and the water content modified by judgement and local practices. Settlement estimates for foundations bearing upon these deposits have been based upon past experience.

The results of pressuremeter tests performed within these deposits allow geotechnical engineers to calculate bearing capacity and make settlement predictions on a more rational basis. The ultimate bearing capacity is a function of the limit pressure of the soil and the settlement is a function of the modulus of elasticity. Both of these values can be measured in these deposits, even though the soil may range from a silty to a cohesive granular texture, with or without gravel particles in the soil matrix.

Project Information

Within the Chicago area, a very dense till deposit is present at a depth of about 80 feet (24 meters) below the ground surface with bedrock being located 20 feet (6 meters) deeper. This deposit (locally referred to as "hardpan") is classified as a silty clay to clayey silt containing varying proportions of sand and gravel with occasional cobbles and boulders. The unconfined compressive strength of this deposit is greater than 7 tsf (6.7 kg/cm^2). The unit dry weight is on the order of 125 to 133 pcf (2.00 to 2.13 kg/cm^3) and the natural water content is generally in the range of 10% to 15%.

Most of the high-rise buildings within the range of 10 to 75 stories in height are supported upon drilled piers extended to the top of this hardpan deposit. Two particular buildings have been selected to illustrate the value of the pressuremeter in allowing settlement predictions to be made for such structures. One project involved a 50-story building with column loads ranging from 2000 to 3000 kips (907 to 1360 metric tons) and a net design soil bearing pressure of 12 tsf (11.2 kg/cm^2). The second project consisted of a 75-story structure with column loads of 16,000 kips (7250 metric tons) and a net design soil bearing pressure of 14.5 tsf (13.9 kg/cm^2).

Project Results and Conclusions

Settlements of the foundations were calculated by assuming that each drilled pier was sufficiently far from the adjacent pier base to be considered an isolated foundation. It was assumed that nearly all settlement would occur in the hardpan and not in the firm bedrock. The calculations were based upon Menard's Formula for settlement, as follows:

$$W = W_2 + W_3 = \frac{1.33}{3E_d} (P)Ro\left(\lambda_2 \frac{R}{R_o}\right)^{\alpha} + \frac{\alpha P}{4.5 E_d} \lambda_3 R \qquad \text{(B-1)}$$

Where:

P = Mean contact stress added to the soil by the rigid footing, kg/sq.cm

R_o = Reference length equal to 30 cm

E_d = Pressuremeter modulus of the soil, an average value was taken of the hardpan tests at the bearing elevation kg/sq.cm.

α = Coefficient dependent upon the nature of the soil and the ratio of E/P

λ_2 λ_3 = Shape of foundation coefficient

R = Radius of foundation, cm

An example calculation for the 50-story structure is as follows:

$$W = \frac{1.33}{(3)\ 405\ \text{kg/sq.cm.}} \quad (11.7\ \text{kg/sq.cm})\ (30\ \text{cm})\ \left(1.12\ \frac{170\ \text{cm}}{30\ \text{cm}}\right)^{.5}$$

$$+ \frac{.5}{4.5\ (405\ \text{kg/sq.cm})}\ (11.7\ \text{kg/sq.cm})\ (1.1)\ (170\ \text{cm})$$

$$W = .97 + .60\ \text{cm} = 1.57\ \text{cm} = .62\ \text{in.}$$

The following table summarizes the observed and pressuremeter predicted settlements.

TABLE 4 - SETTLEMENT OF 50 & 75 STORY BUILDINGS

Location (1)	Net Soil Bearing Pressure in Tons Per Square Foot (2) (2)	Settlement in Inches		
		Pressuremeter Prediction (3)	Measured Range (4)	Measured Average (5)
75 Story Building				
Tower	14.5	2.00	1.69 - 2.19	1.94
Perimeter	10.0	1.25	1.13 - 1.63	1.37
50 Story Building				
Tower	12.0	0.62	0.48 - 0.54	0.50

Note: 1 ton per square foot = .96 kg/sq.cm
1 inch = 2.54 cm

Summary

The close agreement between the predicted and measured settlements illustrates the usefulness of the pressuremeter tests and the semi-empirical rules of calculation developed to estimate settlements of preconsolidated cohesive soils.

C. SUGAR STORAGE SILO

BEARING CAPACITY EVALUATION

General

In stratified cohesive soil deposits, there are frequently layers of softer clay soils interbedded with firm or consolidated soils. When heavy loads are supported upon these deposits, the bearing capacity and compressibility of the softer formations is of vital concern. In most cases, conventional laboratory tests, including unconfined compression and consolidation tests are adequate to determine the properties of these deposits. However, there are occasions when these deposits contain a high proportion of sand and/or silt pockets or seams and the results of the conventional tests may not be representative of the entire soil stratum. Pressuremeter tests performed within these deposits frequently yield different parameters than one would assume from the conventional tests. Usually, the pressuremeter indicates a higher shear strength and lower compressibility than one would assume from the conventional tests and frequently, this makes the difference as to whether the project is feasible.

Project Information

A 116 foot (35 meter) diameter sugar silo was planned for an area where a low strength sandy silty clay was present within the depth range of 35 to 72 feet (11 to 22 meters) below the ground surface. The soils above and below this level were firm and relatively incompressible clayey soils. The silo was to be supported on a mat foundation, except for a ring wall to support the perimeter wall. The unit contact pressure at the base of the silo was 4800 psf (2.3 kg/cm^2).

Two and three inch diameter shelby tube specimens were obtained in the low strength clayey deposit and the unconfined compressive strength was determined to be in the range of 0.4 to 0.9 tsf (0.4 to 0.9 kg/cm^2). The highest strength samples had water contents in the range of 15% to 20% and the lower strength

samples had water contents in the range of 25% to 32%. Conventional bearing capacity theory based upon shear strengths from the confined compressive strength results indicated that a factor of safety of less than 3 would result when the silo was fully loaded. Because of the risky nature, additional borings were made at the site using pressuremeter tests within the low strength clayey despoits.

Results and Analysis

The shear strength from the pressuremeter tests were calculated by the following formula:

$$P_l - P_o = 5.5\ c \qquad \text{(C-1)}$$

A summary of the shear strength, as determined from the laboratory unconfined compression tests and calculated from the pressuremeter tests, is summarized in Table 5 as follows:

TABLE 5 - COMPARISONS OF LAB & PRESSUREMETER TESTS ON SHEAR STRENGTH - LAYERED SOILS

		Shear Strength in tsf	
Soil Strata	Depth in Ft.	Laboratory Unconfined Test	Pressuremeter Field Test
Preconsolidated Silty Clay Crust	15	11.8+	14.0
	29	3.2	3.9
Normally Silty Clay	44	0.8	2.6
	54	0.8	2.5
	68	0.7	3.4

Notes: 1 Foot = 0.30 Meter
1 tsf = 0.96 kg/cm^2

The undrained shear strengths calculated from the pressuremeter tests, are significantly above those from the laboratory tests and are as much as three to four times greater. This is significantly greater than most correlations between shear strength by pressuremeter testing and conventional testing. It may be possible that the higher strength is due, in part, to the presence of the sand and silt content within the clay or thin silt and sand seams. The presence of a high sand content or silt and sand seams within the laboratory samples could significantly affect the strength results.

The pressuremeter test results were also used in computations of the bearing capacity for the silo structure. Conventional analysis using the bearing capacity formula was determined using strength values from the pressuremeter test as well as from the conventional laboratory tests. The results were compared to the bearing capacity using the Menard Pressuremeter Formula as follows:

$$q_{ult} = K (P_l - P_o) + q_o \qquad \text{(C-2)}$$

Where 'K' equals the "depth of foundation factor" which, in this case, is taken as 0.8 for a shallow mat foundation. Menard (reference 9) defines values of 'K' for various types and depths of foundations.

TABLE 6 - BEARING CAPACITIES COMPARISONS

BASIS OF ANALYSIS	FACTOR OF SAFETY
Using Conventional Analysis & Pressuremeter Determined Shear Strength	5.0
Using Conventional Analysis & Average Laboratory Shear Strength	2.8
Using Pressuremeter Formula & Pressuremeter Shear Strength	3.6

Settlement analysis by the pressuremeter formula predicted about 0.3 feet (0.1 meter) of settlement with about 30% of the value as differential between the center and exterior ring wall foundation. Previous predictions based upon conventional analysis estimated 1.0 feet (0.3 meter) of settlement near the center of the silo.

Summary

The higher in-situ shear strength predicted by the pressuremeter test results resulted in an adequate calculated bearing capacity of the structure. In addition, the predicted settlement was within the tolerable range and hence, construction was initiated. The results to date indicated that the settlement is approximately the amount predicted by the pressuremeter theory.

D. STABILITY ANALYSIS OF A ROADWAY EMBANKMENT SLOPE

General

An underground parking garage was constructed adjacent to an existing highway embankment. To construct building foundations and a grade beam parallel to the embankment, it was necessary to steepen the slope of the embankment to 1:1 (horizontal to vertical). Before the work was started, a slope stability analysis was performed and it was determined that the slip circle for deep seated failure was in silty clay roadway embankment fill. Pressuremeter testing was suggested to evaluate the shear strength of the silty clay fill, as unconfined compressive strength tests were not considered to be the most representative of the shear strength of the fill. The in-situ pressuremeter test stresses a larger area and would give a more representative value of the shear strength of the potentially variable fill deposits.

Project Information

To construct caissons and grade beams adjacent to the embankment, a 28 foot (8.5 meter) deep cut into the side of the embankment was required. The geometry of the cut and the soil conditions are shown in Figure D-1. Pressuremeter tests were performed in the silty clay fill deposit encountered between elevation +14 feet and -18 feet, Chicago City Datum (CCD). Three of the tests were performed between elevation +10 and elevation 0 CCD, in the lowest strength zone.

A test pit was excavated into the side of the proposed cut to obtain visual information with regard to a 1:1 slope. Due to a buried pipeline, the test pit had to be sloped to a 1.5:1 (horizontal:vertical) average slope. The sides of the slope appeared fairly stable, except for some minor sloughing and ravelling of fill deposits above elevation +20 feet CCD, where the fill was primarily granular.

Results

The pressuremeter test results in the critical clay fill zone and shear strengths are summarized in the following table.

TABLE 7 - PRESSUREMETER DATA SUMMARY - CLAY FILL SOILS

Elevation (ft)	P_l (tsf)	P_o (tsf)	E/P_l	c (tsf)
+ 10	2.00	0.18	7.5	0.36
+ 5	3.75	0.25	7.5	0.69
+ 0	3.25	0.25	7.7	0.59
Average	3.00	0.23	7.5	0.55

Note: 1 tsf = 0.96 kg/cm^2

The shear strength is calculated by the experimental relationship of:

$$P_l - P_o = 5.1\ c \qquad \text{(D-1)}$$

The coefficient of 5.1 is slightly less than predicted by Menard's theory. According to Menard, the coefficient should be 5.4, which is based upon E/P_l ratio. The experimental relationship was obtained by comparing values of shear strength obtained from unconfined compressive strength tests from previous jobs in the same geographic area.

The average shear strength from the pressuremeter tests was taken to be .5 tsf (.5 kg/sq.cm). The other shear strength parameters and unit densities for the various fills, sand and underlying natural clay layers are summarized on Figure 8 . The stability analyses were performed on a computer using the Simplified Bishop Method of Slices analysis. Two cases were analyzed, the first being the proposed 1:1 (H:V) cut and the second being a 1.5:1 cut. The safety factors were 1.53 and 1.58, respectively.

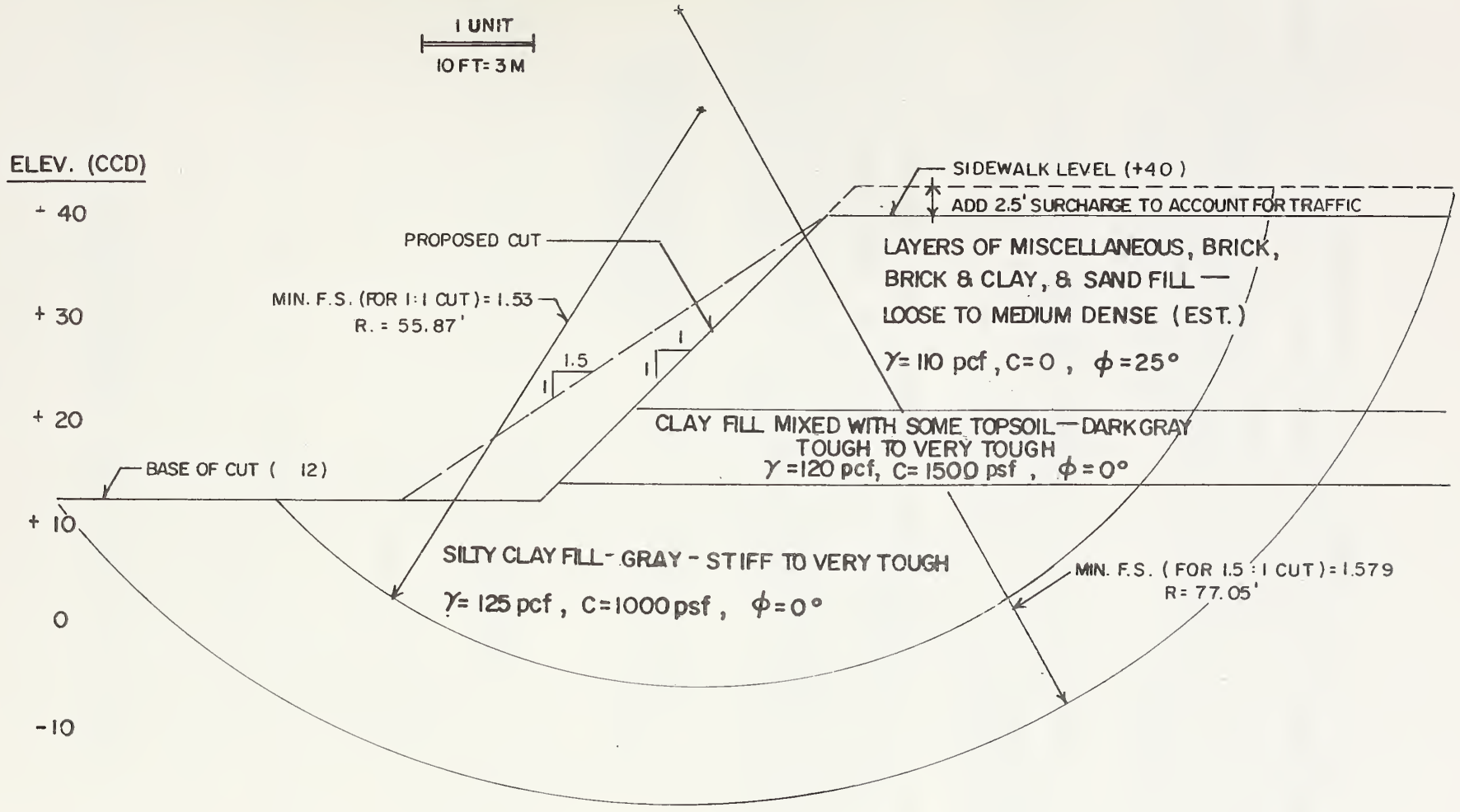

NOTE : 1 FT = 0.3 m
100 PCF = 1.60 Kg/cm²
SCALE : HORIZ. & VERT. ARE THE SAME
1 UNIT
10 FT = 3 M
ELEV. (CCD)
+ 40
+ 30
+ 20
+ 10
0
-10
SIDEWALK LEVEL (+40)
ADD 2.5' SURCHARGE TO ACCOUNT FOR TRAFFIC
PROPOSED CUT
MIN. F.S. (FOR 1:1 CUT) = 1.53
R. = 55.87'
LAYERS OF MISCELLANEOUS, BRICK, BRICK & CLAY, & SAND FILL — LOOSE TO MEDIUM DENSE (EST.)
γ = 110 pcf, C = 0, φ = 25°
1.5
1
1
1
CLAY FILL MIXED WITH SOME TOPSOIL — DARK GRAY TOUGH TO VERY TOUGH
γ = 120 pcf, C = 1500 psf, φ = 0°
BASE OF CUT (12)
SILTY CLAY FILL - GRAY - STIFF TO VERY TOUGH
γ = 125 pcf, C = 1000 psf, φ = 0°
MIN. F.S. (FOR 1.5 : 1 CUT) = 1.579
R = 77.05'

FIGURE 8 - SLOPE STABILITY PROFILE - CLAY FILL SOILS

It was recommended that the 1:1 slope be utilized. Some shallow surface sloughing was anticipated in the miscellaneous fill zone. It was recommended that this zone be stabilized by driving short soldier piles and using wood lagging for support. Also, the cut slope was to be blanketed with polyethelene sheets to minimize surface ravelling and erosion due to drying, wind, precipitation and surface runoff.

As a further precaution, it was recommended that the slope be monitored by inclinometers to evaluate lateral movement which may occur. Figure 9 is a plot showing lateral movement of the slope toward the excavation.

The excavation and slope were stable during construction of the building foundations and grade beams. The pressuremeter shear strength utilized in the stability analysis accurately predicted the soil parameters for use of the slope stability analysis. The movements and settlements were consistent and expected for the safety factors that were computed.

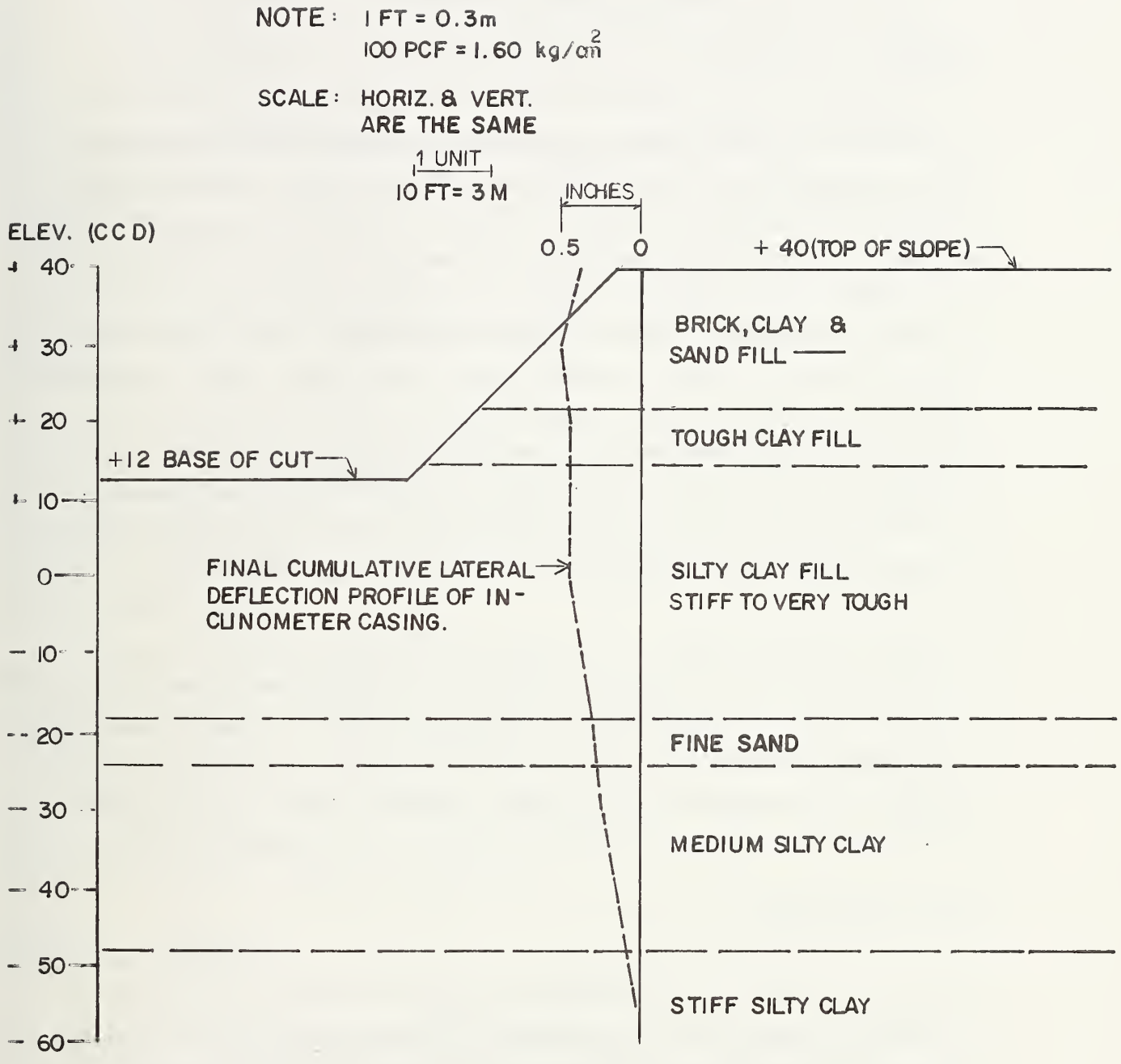


FIGURE 9 - INCLINOMETER MOVEMENT TOWARD EXCAVATION THROUGH CLAY FILLS

E. SHOPPING CENTER CONSTRUCTED

IN AN OLD REFUSE LANDFILL

General

In old landfill sites, the classification and relative density of the fill materials is quite heterogeneous. After a sufficient length of time, the compressible materials have partially or totally decomposed, leaving the relatively inert substances plus fine grained materials, which are the results of the decomposition. The inert substances consist of glass, metal fragments, rubber products and masonry, including brick. There are also pieces of wood or leather which, given a sufficient length of time, would also become decomposed. Determining the properties to predict behavior of these materials under loads is very difficult. The conventional means of subsurface exploration, such as split-barrel sampling or the thin wall shelby tube sampling are not reliable. The large particles within the fill affect the penetration resistance value and there is not enough cohesion for these particles to hold together when extruded from the shelby tube.

To obtain design parameters for hetrogeneous fills, in-situ testing is necessary. The pressuremeter is an excellent tool that can be used to determine fill properties for bearing capacity and settlement predictions. A larger number of tests is required to determine the average properties of fill as compared to the number of tests required for normal soil deposits. This is because of the extreme variations that can occur in both horizontal and vertical directions.

Project Information

A former clay pit near the western edge of a large metropolitan area was used as a landfill site. The clay mining extended to a level ranging from 40 to 60 feet (12.2 to 18.3 meters) below prevailing grade. Refilling of the pit started in 1930 and was completed in 1949. All types of materials were dumped into the fill because of the lack of zoning restrictions during that time. The landfill included

ordinary refuse, tree stumps, broken concrete and asphaltic materials from pavements, clay from nearby excavations, cinders from coal buring and masonry, including brick from demolished buildings. The site remained undeveloped from 1949 to 1975. The ground surface was irregularly shaped and a profile through the site would resemble a sine wave with about 2/3 of the site filled to a height as much as 100 feet (30.5 meters) above the prevailing grade. The remaining 1/3 of the site had no landfill placed, so the ground was 60 feet (18.3 meters) below the prevailing grade. In 1975, the land was investigated for development as a shopping center.

The investigation included pressuremeter tests in the existing landfill materials plus pressuremeter tests performed within test strips of compacted site materials.

The pressuremeter tests indicated that in the existing landfill, the limit pressure and pressuremeter modulus varied considerably, but generally both values improved with increasing depth. This was anticipated since the deposits can be considered to be normally consolidated and the degree of consolidation increases with increasing depth.

The pressuremeter tests performed in the compacted test strips indicated limit pressures on the order of 10 to 13 tsf (kg/cm^2) and modulus values on the order of 125 tsf (kg/cm^2) were possible if the material was placed as a controlled compacted fill. Project specifications required lifts not exceeding 1.0 feet (0.3 meter) thick and compacted to a minimum of 95% of the maximum density, determined by ASTM D-1557 test procedures.

Conclusions and Results

Primarily, on the basis of the pressuremeter test results, it was concluded that the shopping center project could be constructed utilizing shallow spread footing foundations. The top of the existing landfill was to be cut, placed and compacted in low ground where no landfill existed. Pressuremeter tests performed within the compacted backfill indicated limit pressure values and modulus values

comparable to those which were achieved with the test section. In the cut areas, the limit pressures at the final grade were 5 to 7 tsf (kg/cm^2) with modulus values on the order of 60 to 80 tsf (kg/cm^2). These values are indicative of a medium dense condition that was achieved because of the surcharge effect of the top of the hill which was removed to fill in the low ground.

In the transition areas where there was neither significant cut or fill, limit pressure values of 3 to 5 tsf (kg/cm^2) and modulus values on the order of 25 tsf (kg/cm^2) were measured with the pressuremeter test.

In the transition areas, the foundation support conditions were improved by densifying the soils through dynamic compaction. The pressuremeter was used to determine the effectiveness of the dynamic compaction operations. The densification continued until a minimum limit pressure of 5.5 tsf (kg/cm^2) and a minimum modulus of 50 tsf (kg/cm^2) was obtained below the proposed footing level. These values were selected on the basis of bearing capacity considerations and a limiting predicted differential settlement between adjacent columns of 1/2 inch (1.3 cm).

The alternative to using spread footings would have been to extend the foundations by means of drilled caissons or piles to levels of 50 to 80 feet (15.3 to 24.4 meters) below the floor slab grade. This foundation alternative would have rendered the project to be non-feasible from an economic standpoint.

The first phase of the shopping center was completed in 1976 on the compacted portions of the landfill. Settlements were found to range from near 0 to 5/8 inch (0 to 1.5 cm). This agrees with the settlement predictions. The second phase of the project in the cut and transition areas is currently under construction and long term settlement readings are not available.

F. DRILLED CAISSON LOAD TESTS

General

Drilled caisson foundations are frequently utilized to support heavy structures. Many caisson foundations are designed strictly as end-bearing and a bell may be constructed at the bottom to enlarge the bearing area. Any contribution of side friction to the total bearing capacity is typically neglected along the shaft of a belled caisson where the shafts extend through low strength deposits. In some instances, straight shaft caissons are extended into hard, dense soil but because of the inability to construct bells in this material, or because high end bearing capacities in deeper strata are possible, a bell is not needed. In these instances, side friction in the dense soils can significantly increase the load capacity.

In order to obtain information about end bearing and side friction for caissons, three caisson load tests were conducted on the near-south side of Chicago. The caissons were loaded as 1) end-bearing, 2) side friction only and 3) end-bearing and side friction. Pressuremeter tests were performed in borings drilled near the test caissons to compare predictions of settlement and bearing capacity with what was observed in the caisson load tests. Caisson load tests presented an excellent opportunity to correlate actual foundation performance with regard to Menard's formulas.

Project Information

The soil strata at the site consisted of fine sand to a depth of 20 feet (6.1 meters), which was underlain by moderate to low strength silty clay to a depth of approximately 51 feet (15.6 meters). Hard silty clay till with occasional layers of dense and extremely dense silt and sand was present below the low strength clay. This zone extended to the top of the rock surface, approximately at a depth of 70 feet (21 meters). The strength of this layer increased with depth.

Test Caisson Arrangement & Instrumentation

The bearing caisson was achieved by augering a 2.5 feet (0.7 meter) diameter shaft full depth to the bearing elevation, 51 feet (15.6 meters) and pouring a short concrete plug (1.3 feet, 0.4 meter) setting a 2 foot (0.6 meter) diameter corrugated metal casing on the concrete plug and filling the casing with concrete and then filling the annular space between the soil and the casing with bentonite slurry. The friction caisson consisted of a 2 foot (0.6 meter) diameter shaft with a void at the bottom of the hole at the desired bearing elevation. The void was created by means of a plywood board about the same diameter of the shaft and an inflatable doughnut-shaped innertube, which had sufficient air pressure to support the head of concrete. However, the validity of this method was in doubt and the results not given high significance in the analysis which follows. The combined end-bearing and friction caisson was a 2 foot (0.6 meter) shaft that was augered to the bearing level and the entire shaft filled with concrete.

The caissons were instrumentated by installing telltales in order to measure the movement of the bottom of the caisson, relative to the top. Deflection gages were used to measure the movement of the caisson bearing plate and telltales. Caisson load test results are shown in Figure 10 .

Results and Conclusions

Pressuremeter tests were performed in borings near the test caissons above the bearing stratum as well as in the bearing stratum.

Shear Strength

The pressuremeter predicted shear strength in the general soil profile agreed with the laboratory determined unconfined compressive shear strength, although it was as much as 33% higher in the bearing stratum. This may be the result of the in-situ nature of testing; laboratory samples are typically slightly disturbed especially for very hard soils.

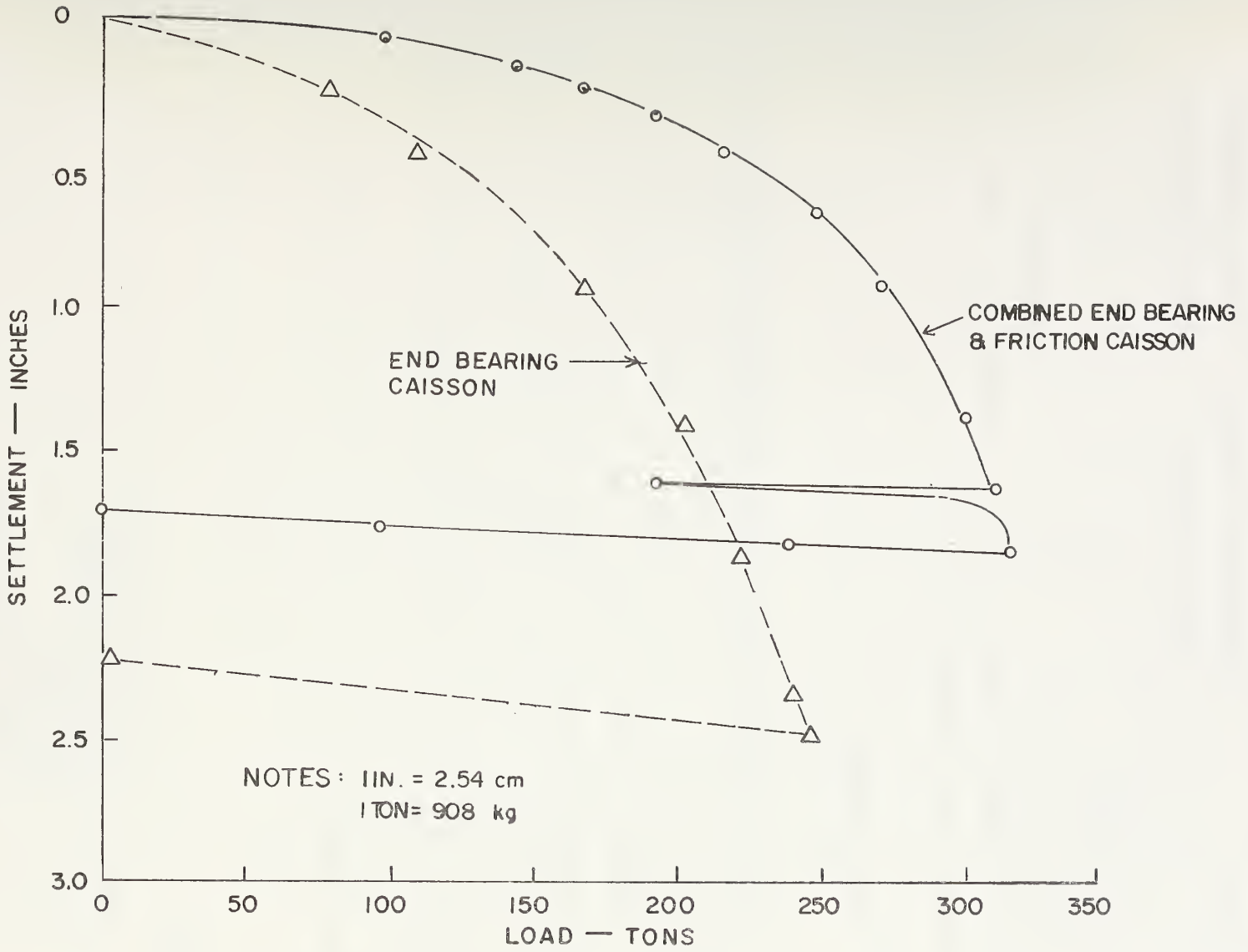


FIGURE 10 CAISSON LOAD TEST RESULTS

TABLE 8 - COMPARISON OF LAB & PRESSUREMETER TEST
SHEAR STRENGTH - GLACIAL FILL & HARDPAN

Soil Strata	Pressuremeter Predicted Shear Strength (tsf)	Average Unconfined Shear Strength (tsf)
Very Stiff Silty Clay	0.8	0.6
Hard Silty Clay	2.8	2.9
Silty Clay Hardpan	7.2	6.0

Note: 1 tsf = 0.96 kg/cm^2

Bearing Capacity

The capacity of the end-bearing caisson was calculated by conventional theory and by Menard's formulas. Conventional theory used in the Chicago area is based upon Skempton's research, which indicated the ultimate bearing capacities should be about nine (9) times the cohesion. For cohesion taken as one half of the unconfined compressive strength of the soil, the theory would indicate a maximum end-bearing capacity of about 25% below that noted in the load test, when the unconfined compressive strength is averaged in the zone 3 diameters above and 3 diameters below the bearing level. When the unconfined compressive strength at the bearing level only was used, the predicted bearing capacity was identical with the capacity noted in the field test.

Menard's bearing capacity theory is based upon the following formula:

$$q_{ult} = K(P_l - P_o) + q_o$$

where:

q_{ult} = ultimate bearing capacity
K = bearing capacity factor
P_l = limit pressure
P_o = at-rest horizontal pressure
q_o = overburden pressure

For a bored pile, the bearing factor would be 3.2 according to Menard. Also, it is necessary to determine an equivalent limit pressure, based upon the soil within the zone 3 diameters above and 3 diameters below the bearing level. Based on these two considerations, the ultimate bearing value predicted by Menard was about 77% greater than that noted in the load test. Backfiguring a bearing factor results in k = to 2.0 rather than 3.2. This reduction in k tends to correlate with Menard's approach in that the confinement of the foundation is critical to the bearing capacity factor. The end-bearing caisson was constructed with a lack of confinement in the annular space around the caisson shaft. When the Menard formulas are recalculated assuming a limit pressure of 1 above the bearing level, the equivalent limit pressure calculated results in an ultimate bearing capacity of about 27% below that realized in the field load test.

The results of the bearing capacity predictions are shown in Table 9 .

TABLE 9 - CAISSON LOAD TEST RESULTS

FIELD LOAD TESTS	END BEARING CAISSON (2.5 ft. dia.) (1)	FRICTION CAISSON (2.0 ft. dia.) (2)	FRICTION PLUS END BEARING (2.0 ft. dia.) (3)
Ultimate Load	275 tons (56 tsf)	164 tons (1.2 tsf)	340 tons
BEARING CAPACITY EVALUATION			
Conventional Theory ($q_{ult} = 9c$)			
1) c from ± 3 diameters of bearing level	46 tsf	-	-
2) c at bearing level only	54 tsf	-	-
Pressuremeter Theory ($q_{ult} = q_o + K[P_l - P_o]$)			
3) Using equivalent P_l from ± 3 diameters of bearing level K=3.2 for drilled pier	104 tsf	-	-
4) Using equivalent P_l from 0 to -3 diameters of bearing level, K = 0.8 for "surface foundation"	39 tsf	-	-
SIDE FRICTION EVALUATION			
Conventional Theory ($S_o = 2\pi R(C_a)$)			
1) For total friction mobilization $C_a = 1.0$	-	413 tons	-
2) For probable friction mobilization, $C_a = 0.5$	-	206 tons	-
Pressuremeter Theory (From Menard Charts)			
3) Along full shaft length	-	157 tons	-
4) Along full shaft length excluding zone 0 to +3 diameters above bearing level	-	111 tons	-
COMBINED CAISSON ANALYSIS			
Field Load Test	275 tons (56 tsf)	164 tons (1.2tsf)	340 tons
Conventional Theory	170 tons (54 tsf)	206 tons	376 tons
Pressuremeter Theory	123 tons (39 tsf)	157 tons	280 tons

NOTE:

1 ft = 0.305 m
1 ton= 8.9 kN
1 tsf= 95.8 kPa

G. DEEP EXCAVATION IN SOFT SOIL

General

At a site in downtown Chicago, a 31 foot (9.4 meters) deep unbraced excavation was performed, which resulted in significant movements of the cut soils. Pressuremeter tests were performed at the base of the excavation in order to aid in evaluating the shear strength of the soil to allow computation of excavation stability. These tests were performed for pressuremeter research to help correlate with other methods of determining shear strength: in-situ vane shear and laboratory tests on large diameter undisturbed samples.

Project Information

The soil conditions at the site generally consisted of surface building rubble fill overlying a thin crust of hard, silty clay, which overlies the soft clay in which the excavation base was situated. Dense silt, hardpan and limestone bedrock lie below the soft clay starting at about 20 feet (6 meters) below the excavation level. There was a perched water table at the site located in the upper fill between 7 and 10 feet (2 to 3 meters) from the original grade.

The soils were glacial in origin. The thin clay crust has been preconsolidated by desiccation and fluctuations in previous water levels. The soft clay is generally found to be normally or just slightly preconsolidated. It has been found to be uniform in index properties over short horizontal directions.

Results and Conclusions

The base of the excavation was in the soft to medium stiff silty clay. The silty clay had an average water content of 23% and an average moist unit weight of 128 pcf (2.05 kg/cm^3). Laboratory consolidation tests indicated the soil to be normally consolidated; the coefficient of horizontal earth pressure, K_o, was determined to be 0.55.

Pressuremeter tests were performed at the base of the excavation in shallow bore holes advanced by hand auger methods. The shear strengths calculated from the pressuremeter tests, were compared to shear strength values from field vane shear tests and laboratory unconfined compressive strength tests. The pressuremeter test results were excellent, considering the low strength of the clay soils. They are shown in Figure 11 . Typically, pressuremeter tests performed in very soft deposits by drilling or augering, result in poorly defined curves due to bore hole disturbance.

The shear strength was calculated by the theoretical relationship according to Menard:

$$P_l - P_o = 2K_b c \qquad \text{(G-1)}$$

The coefficient, K_b, is usually determined from the $E/(P_l)$ ratio. For typical cohesive soils, this ratio equals 10, for which the value for K_b is 2.7, according to Menard. However, this value was modified as based on relationships established for other Chicago area clays; a value of 2.55 was used. The horizontal earth pressure at-rest (P_o) was determined from the laboratory K_o value and compared with the test curve value taken as the start of the pseudo-elastic portion. For example, the shear strength calculated from the pressuremeter Test 1 is as follows:

$$(P_l - P_o)/2K_b = (2.75 \text{ kg/sq.cm.} - 0.70 \text{ kg/sq.cm.}) / 5.1 = 0.40 \text{ kg/sq.cm.}$$

The vane shear tests were all performed in the soft silty clay strata and at the same depth as the pressuremeter test. The vane shear tests were performed approximately 200 feet away. Five (5) inch (12.7 cm) diameter thin wall steel tubes for unconfined compressive strength tests were also obtained within 200 feet of the pressuremeter test hole at the same depth at which the tests were run in the soft silty clay strata. The index soil properties of the soft silty clay strata at this site were uniform with depth, showing no major significant variations, so that the correlation between test results is possible. All of these tests, pressuremeter, vane shear and large sample unconfined compression, were performed from the excavation level. The shear strength data is summarized in the following table.

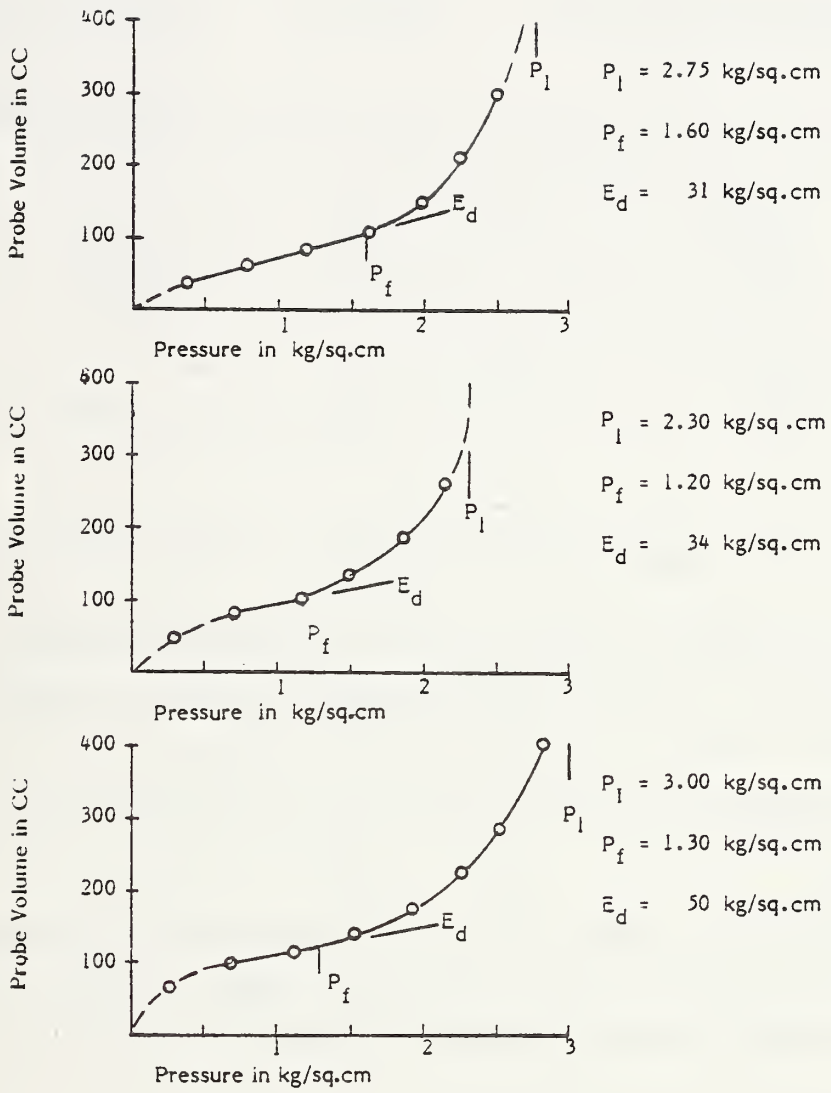


FIGURE 11 - PRESSUREMETER DATA - SOFT CLAYS

TABLE 10 SUMMARY OF SHEAR STRENGTH RESULTS

Pressuremeter Predicted Shear Strength, tsf	Vane Shear Tests, tsf	Unconfined Compression Test Shear Strength*, tsf	
		@5% Strain	@Max. Stress
.40	.44	.40	.55
.31	.31	.25	.45
.45	-	.40	.64
Average			
0.39	.38	.35	.55

Notes: 1 tsf = .96 kg/sq.cm.

* From 5 inch (12.7 cm) diameter thin wall tube samples for strain

All values of shear strength for the three methods were relatively close. The two methods of testing in-situ were higher than the values predicted from the laboratory testing at 5% strain; the higher laboratory values at maximum stress tend to indicate that the soil would yield in a plastic shear, as was actually noted in the excavation cut.

The design shear strength used to calculate base stability for no soil movements within the excavation was conservatively chosen to be 0.3 tsf (0.3 kg/sq.cm). This was also the lowest pressuremeter value of shear strength.

CASE HISTORY

H. LATERAL DEFLECTION OF A TRANSMISSION TOWER

Background

In recent years, it has become common practice to utilize a single foundation element, such as a drilled pier, for supporting a transmission tower. These units are subjected to high overturning forces, which must be resisted by the lateral soil pressure adjacent to the drilled pier. This type of foundation has proven to be more economical than the conventional four footing tower or a large mat foundation. One of the main considerations for the single drilled pier foundation is the ultimate resistance to overturning and the deflection of the pier under the normal overturning force.

The first step in the design of a drilled pier or pile foundation to support overturning loads consisted of determining the ultimate capacity of the foundation. The foundation was then sized to have a sufficient safety factor such that the deflection under a working load would be small.

Some of the early attempts at predicting the deflection were based upon the horizontal subgrade reaction as proposed by Terzaghi (reference 14). However, the horizontal modulus of subgrade reaction could not be easily measured and it was frequently estimated on the basis of soil type and the relative density. Other investigators (references 16 and 17) have predicted the deflection on the basis of triaxial tests and strain measurements and, some degree of success has been obtained in correlating computed values with actual field test data. Recently, the pressuremeter test has been used to evaluate the horizontal modulus of the soil, which, in turn, can be converted to a horizontal subgrade reaction. This information is then used in a deflection analysis.

Project Description

At a site in northeastern United States, two drilled pier foundations were installed and load tested to determine the deflection under lateral loads. One drilled pier was 5.5 feet (1.68 meters) in diameter and the other, 8 feet (2.4 meters) in diameter. Both piers were extended to a level of 17 feet (5.2 meters) below the ground surface. An inclinometer was installed in each of the drilled piers throughout the full length to determine the deflection at various levels below grade. In addition, horizontal deflections were measured at the ground surface using a scale and a fixed reference system consisting of two dead-men plus a taut piano wire. A transmission tower was erected on top of the drilled pier and lateral loads were applied with steel cables. The loads were measured by means of a dynamometer. After the application of each load increment, surface deflection readings were taken on the scale and the deflections were recorded at the various depths by means of the inclinometer readings.

The soil profile at this site consisted of a silty soil containing a trace of clay and particles of decomposed shale. As the borings were made, pressuremeter tests were performed at various levels below the ground surface. Pressuremeter parameters and Standard Penetration Resistance values from two borings are shown in Table 11 .

TABLE 11 - PRESSUREMETER DATA SUMMARY - SILTY SAND SOILS

BORING	DEPTH (ft)	P_f (tsf)	P_l (tsf)	E_d (tsf)	E_d/P_l	"N"
B-1						
(8.0 Ft. Diameter Pier)						
	0 - 2.5	1.7	3.5	38	10.8	5
	2.5 - 5.0	9.0	18.0	200	11.1	24
	5.0 - 7.5	12.0	24.0	320	13.4	26
	7.5 - 10.0	11.0	22.0	275	12.5	28
	10.0 - 12.5	13.5	27.0	380	14.1	26
	12.5 - 15.0	10.0	22.0	200	9.1	29
	15.0 - 17.5	13.0	26.0	270	10.4	30
B-2						
(5.5 Ft. Diameter Pier)						
	0 - 2.5	2.2	4.5	45	10.0	7
	2.5 - 5.0	3.7	7.5	60	8.0	21
	5.0 - 7.5	9.5	16.0	170	10.6	20
	7.5 - 10.0	11.0	21.0	260	12.4	42
	10.0 - 12.5	8.0	16.0	220	13.8	26
	12.5 - 15.0	6.7	13.5	100	7.4	20
	15.0 - 17.5	4.5	9.0	55	6.1	22

NOTE: 1 ft = 0.305 m
1 tsf = 95.8 kPa

Below a depth of 2.5 feet (0.7 meters), the Standard Penetration Resistance value was fairly constant at 20 to 30 blows per foot, but the pressuremeter modulus was found to range from 160 to 260 tsf. The ratio of pressuremeter modulus to Standard Penetration Resistance ranged from 2.9 to 14.6 and averaged 7.6. This is a significant variation. Some analyses for deflection are based upon correlations with Standard Penetration Resistance, which, in this case, would tend to indicate a relatively uniform deposit, but the pressuremeter testing indicates a wide variation in pressuremeter modulus, which represents lateral soil stressing. Terzaghi (reference 14) specifies linearly increasing modulus for granular soils which is confirmed by the pressuremeter test data.

The pressuremeter modulus was converted to horizontal subgrade reaction by the following relationships (reference 15)

$$k \times R = \alpha' = E_s \qquad \text{(H-1)}$$

and

$$E_s = 3E_d \qquad \text{(H-2)}$$

and

$$k = k_o \left(\frac{R}{R_o}\right)^{-3/4} \qquad \text{(H-3)}$$

where:

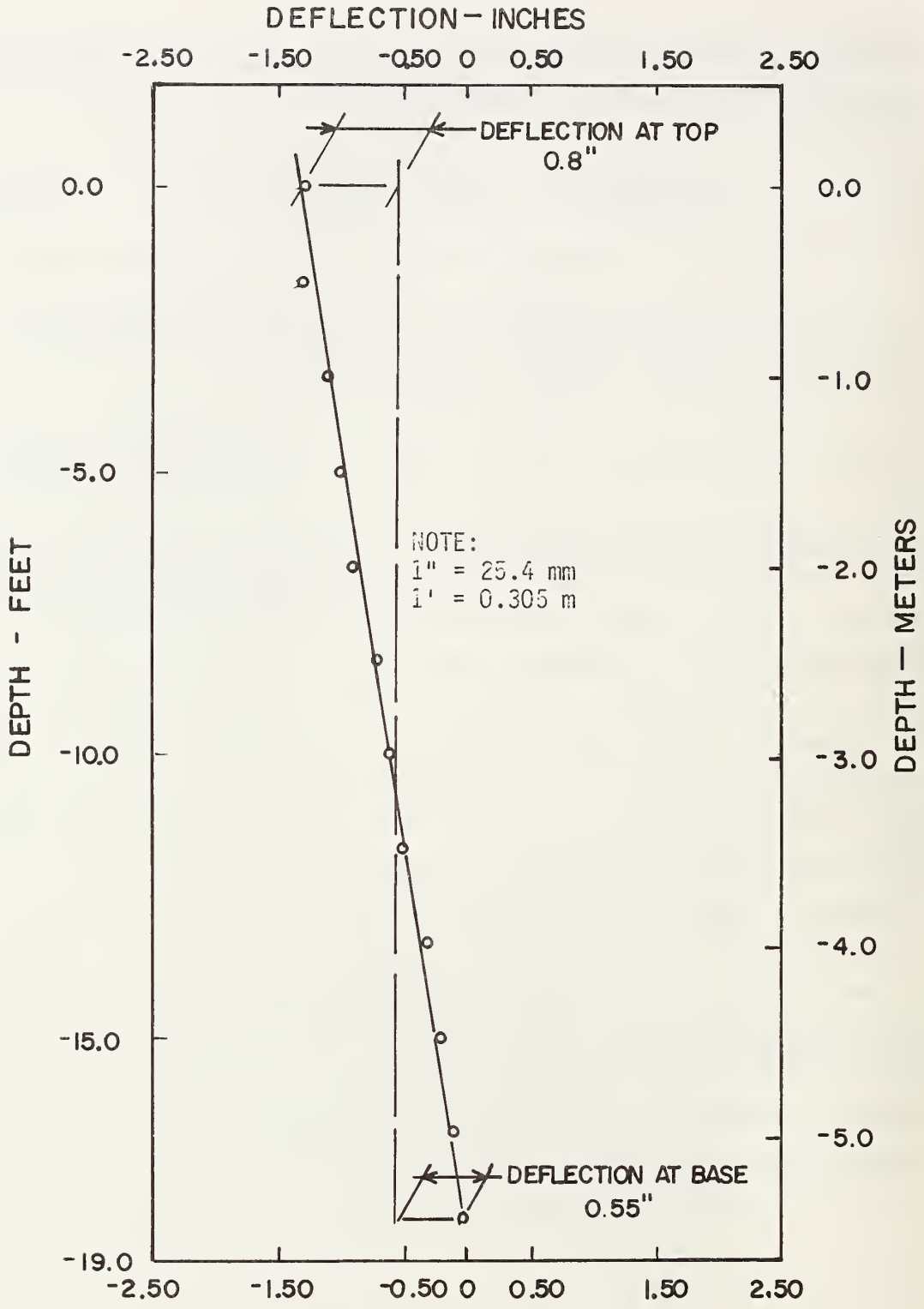
k	=	horizontal modulus of subgrade reaction
k_o	=	horizontal modulus of subgrade reaction for 30cm. diameter plate
R_o	=	standard diameter of 30 cm.
R	=	width of loaded area
α'	=	constant determined by bending characteristics of the pile or pier
E_s	=	modulus of deformation of the soil by plate load test
E_d	=	pressuremeter modulus

The modulus of horizontal subgrade reaction could also be calculated by means of the Menard Formula (reference 9) as follows:

$$1/k = \frac{1.33}{3E_d} \times 30 \times \left(\frac{R}{30} \right)^{\alpha} + \frac{1}{4.5} \frac{\alpha}{E_d} \lambda_3 R \qquad \text{(H-4)}$$

The modulus of horizontal subgrade reaction computed by equations H-1 to H-3 or equation H-4 were found to be in close agreement.

A Beam Column computer program was used to calculcate the deflection, moments and shears at various points within the drilled pier utilizing the horizontal subgrade reaction modulus determined from the pressuremeter tests and the stiffness of the drilled pier. The deflection that is generated by this computer solution is based upon assumptions regarding the boundary conditions at the base; i.e., whether or not the base can translate or whether the slope must remain vertical or can deflect. The inclinometer data from the two drilled piers under loads indicated that the drilled piers remained rigid during load application and translation of the top in the direction of the force plus a reverse movement at the base did occur. A plot of the deflection of the 5.5 foot diameter drilled pier at a maximum load increment is shown in Figure 12. Based upon the deflection behavior of this pier under load, it was decided that the boundary condition that is most representative for the caissons in this load test was the case of base freedom, i.e., the base is free to translate and the slope is free to move from the initial vertical position. This use of boundary condition results in the maximum predicted deflection. Computer predictions for base fixity reduced the deflections predictions by a factor of 2 to 5, depending upon the restrictions imposed.



DEFLECTION OF 5'-6" DIA. DRILLED CAISSON AT MAXIMUM OVERTURNING

FIGURE 12 - PIER DEFLECTION AT MAXIMUM LOAD

Project Results

The measured deflections of the top of the pier under the two cycles of loading that were applied to the 5.5 foot (1.6 meter) diameter pier are shown in Figure 13a. Figure 13b is a plot of the measured deflections under five cycles of loading that was applied for the 8 foot (2.4 meter) diameter pier. The predicted deflection, based upon the pressuremeter derived soil input and the condition of freedom of the base of the pier to translate and rotate are also shown on these two figures. The predicted deflection overestimated the measured deflection, especially for the 8 foot (2.4 meter) diameter pier. Since the diameter was about one half of the height, a considerable amount of restraining moment was available at the base that would help to reduce deflections. However, the computer program was not capable of taking this effect into account. A modification of the computer program appears warranted to handle wide piers. In the case of the 5.5 foot (1.6 meter) pier, the measured and predicted deflections were in relatively close agreement. The effect of base restaining moment for this pier would be interpreted to be significantly less than for the 8 foot (2.4 meter) diameter pier.

Since the predicted and measured values of the surface deflection were reasonably close in agreement, it is concluded that the pressuremeter was an excellent tool for measuring the soil parameters. These parameters were converted to a deformation modulus which, when coupled with a computer program, accurately predicted deflections under lateral loads.

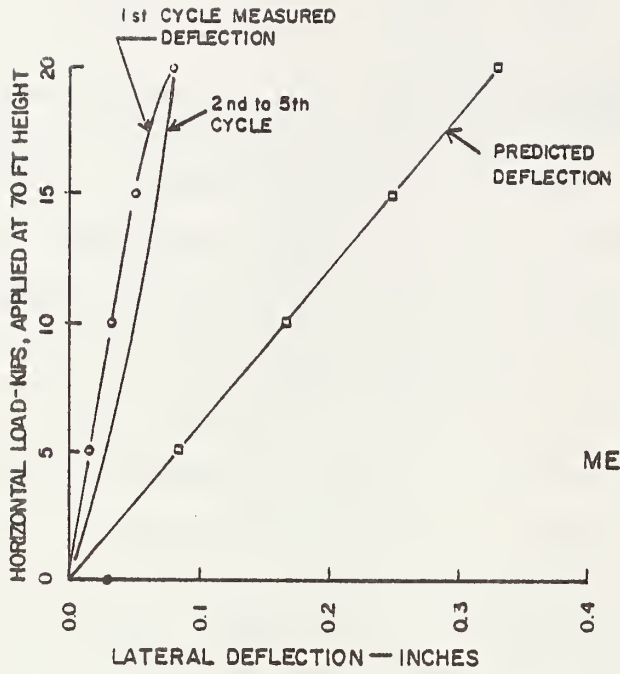


FIGURE 13a
MEASURED & PREDICTED DEFLECTION
8'-0" DIA. PIER

NOTE: 1 kip = 4.45 kN
1 in. = 25.4 mm
1 ft. = 0.305 m

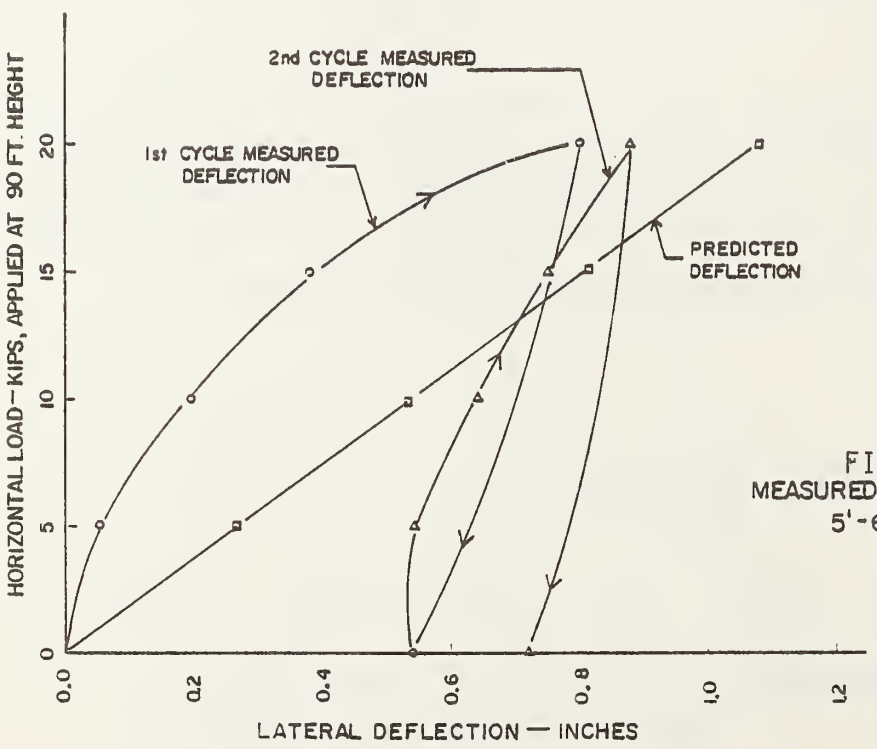


FIGURE 13b
MEASURED & PREDICTED DEFLECTION
5'-6" DIA PIER

FIGURE 13 - CYCLE LOADING OF PIERS

CASE HISTORY

I. LATERAL LOAD TEST FOR CHIMNEY OVERTURNING EVALUATION

"H" pile foundations, driven to support a 500 foot high chimney, were load tested in a lateral direction to evaluate the soil resistance to the large wind overturning moments on the chimney. Pressuremeter tests were performed in conjunction with the four lateral load tests to help provide information regarding horizontal subgrade reaction of the soils. Standard Penetration Resistance Values in the fine to coarse sand soils, as well as near-surface field density tests, were performed to model the soil profile. This information of soil profile, relative density, and modulus of subgrade reaction was input into a "Beam-Column" computer program and the resulting predictions of pile deflections were correlated with the actual field test measurements. The shear stresses and moments predicted by the computer were checked versus allowable values for the type of pile selected.

Test Results

The piles were loaded and unloaded through three cycles, the maximum load being four times the design lateral load. Figure 14 indicates the lateral load test results for an HP-10 X 57 pile. A maximum net deflection of 0.5 in. (1.3 cm) was set as a limit after removal of all loads. All tests were well within this limit.

The deflection during the loading portion of each cycle became larger at the same load, with each new cycle. An increase in deflection should bring about a corresponding reduction in modulus of subgrade reaction. The units for subgrade reaction are typically force per length cubed. More accurately, the units should be force per unit area moving a rigid surface through a specific distance. The larger the value of subgrade reaction, 'k', the greater is the

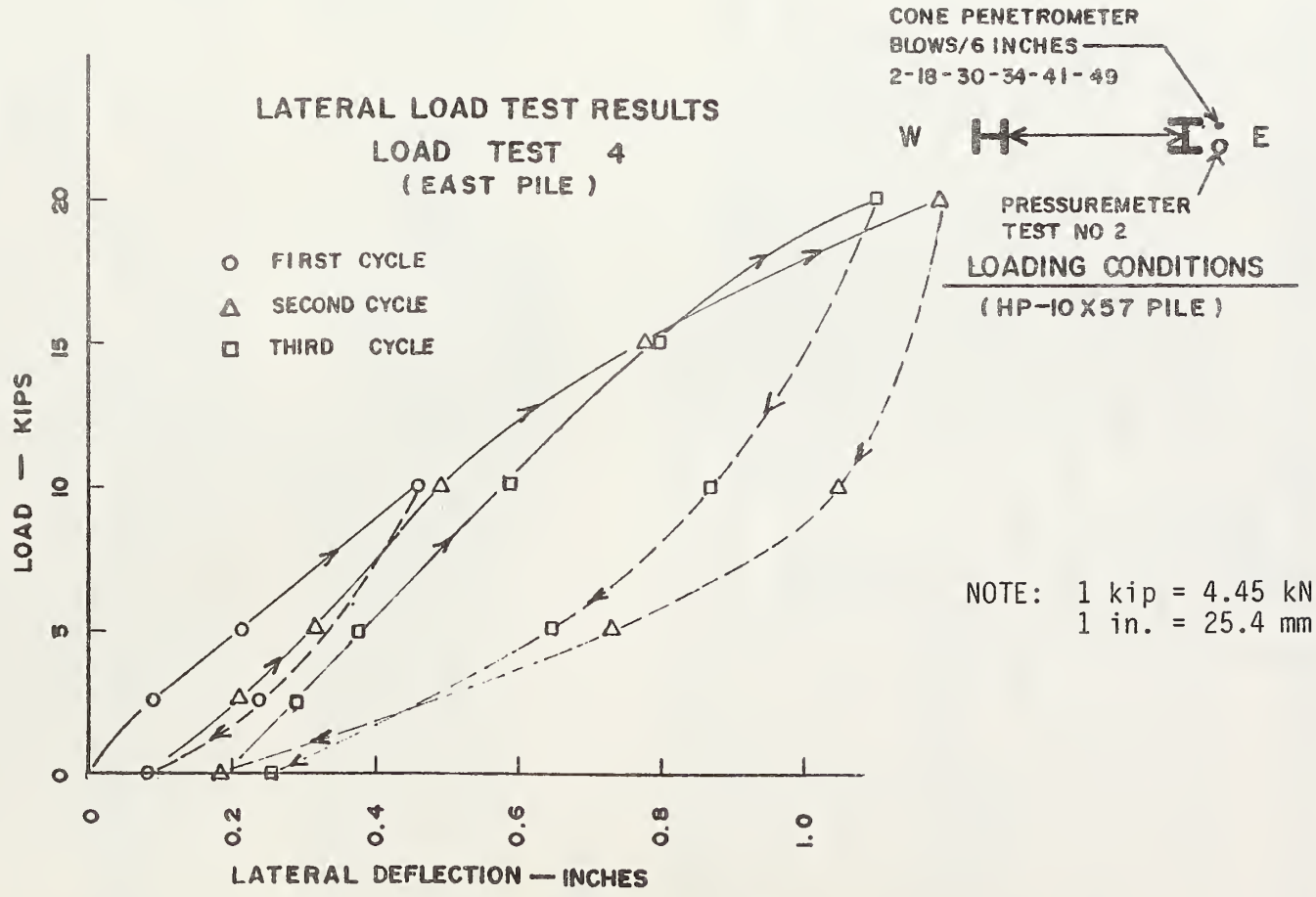


FIGURE 14 - LATERAL LOAD TESTS OF H-PILE

force required to move the rigid surface through a unit distance. Conversely, for a uniform force per unit area, a larger deflection through which the rigid body moves will result in a smaller 'k' value. Thus, as the deflection increases, the modulus of subgrade reaction decreases.

Pressuremeter tests were performed both before and after the load testing. Although they could not be performed in the same bore hole at the same depth, the soil conditions were uniform enough to allow correlations between near-by bore holes. There was a significant reduction, in the pressuremeter modulus, E_d, after the load test was performed, approximately 40%. As defined by Menard (reference 9) and Baguelin (reference 1) the modulus of subgrade reaction is directly related to the pressuremeter modulus; in this case, by a factor of 2.3 times the pressuremeter modulus, E_d.

The computer program was used to define deflection curves for various values of modulus of subgrade reaction. These are shown in Figure 15. By correlating with the actual deflection measured for the pile top (at 1.0 foot (0.3 meters) above grade) the approximate deflection curve can be established. The initial and third cycle deflection curves are shown in heavier lines in Figure 15. The bending in the upper one foot is due to the very loose nature of the surface soil around the pile which was not modeled in the computer program. However, when comparing the pressuremeter test results before and after pile loading at a pressuremeter test depth of between 2 and 4.5 feet (0.6 to 1.4 meters) the corresponding reduction in theoretical modulus of subgrade reaction value corresponds nearly identically with that measured in the pressuremeter test.

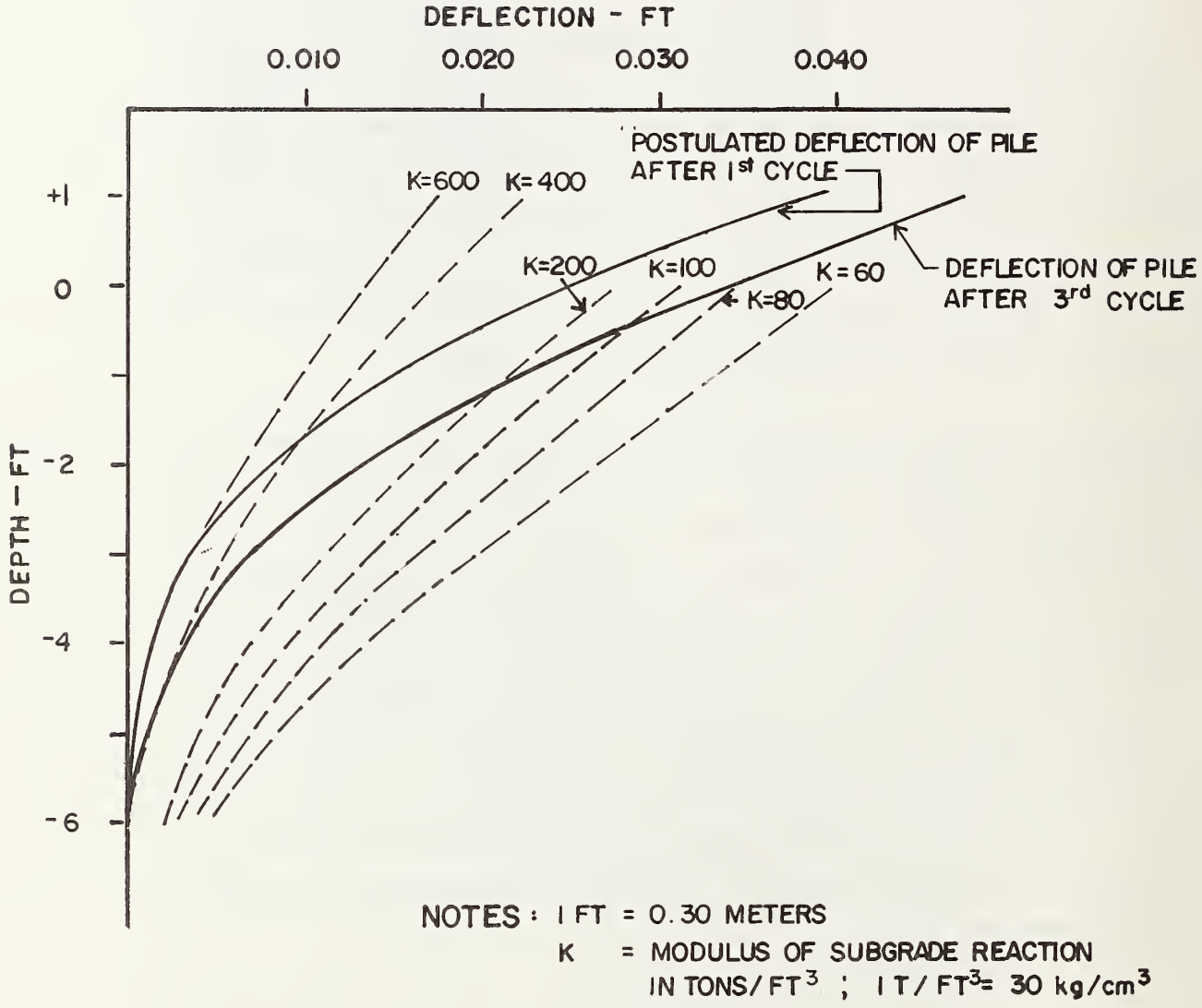


FIGURE 15 - PILE DEFLECTION VS. MODULUS OF SUBGRADE REACTION

Measurement of Horizontal Pressures With the Pressuremeter

Experimental pressuremeter testing was performed during application of test loads. A pressuremeter bore hole was drilled 1.0 foot (0.3 meters) from the flange of the H-pile foundation. A pressuremeter was introduced into the bore hole and a seating pressure was applied to overcome inertia and to stabilize the pressuremeter probe against the sides of the bore hole. As the load test was applied to the pile, the pressure was transmitted to the soil, tending to collapse the pressuremeter probe. A slight pressure was applied to the pressuremeter probe so that a constant volume was maintained. Each time the load was applied to the pile, a corresponding pressure increase was applied to the pressuremeter probe to maintain this constant volume. In this way, the stress distribution into the soil by the loaded pile was measured.

The total test load was applied at the top of the pile, and the reaction to this load was distributed throughout the soil within the full length of the pile. The computer program was used to estimate the approximate stress distribution in the soil. The average load distribution through the length of pressuremeter test (depth of 2 to 4.5 feet (0.6 to 1.4 meters)) was about 21% of the applied total load on the pile. This distribution was somewhat independent of the modulus of subgrade reaction for the pile tested, so changes in modulus of subgrade reaction had little influence of total load distributed to the soil within this zone. The maximum pressuremeter pressure required to maintain a constant volume was approximately 8% of the total applied test load.

Stress distribution in soils from loaded foundations is typically assumed to spread outward in a "stress bulb" fashion. The portion nearest to the foundation resembles a straight line on a slope of approximately 45^{o} to 60^{o}. The reduction in stress from 21% of the total load at the pile surface to 8% one foot (0.3 meter) from the pile represents a stress distribution of approximately a slope of 51^{o} outward from the loaded pile.

Conclusions

In summary, the pressuremeter results can be used to predict modulus of subgrade reaction which, in turn, can be input into an appropriate computer program to predict lateral pile deflections with some degree of accuracy. Changes in modulus of subgrade reaction due to cyclic loading may be discernible by accurate pressuremeter testing.

The pressuremeter may be useful in measuring in-situ stress changes due to stress increases caused by loaded foundations. Additional study and research should be conducted in this area to more accurately define the reactions which can take place.

APPENDIX A - REFERENCES

1. Baguelin, F., Jezequel, J. F. and Shields, D. H., The Pressuremeter and Foundation Engineering, Series on Rock and Soil Mechanics, Volume 2 (1974/77) No.4., 1978 by Trans Tech Publications, Clausthal, Germany.

2. Broise, Y. "Operation Manual - Geoprobe Model GW-H", 1970.

3. Brooker, E. W. and Ireland, H. O., "Earth Pressures At-Rest Related to Stress History", Canadian Geotechnical Journal, Volume 11, No.1, February 1965.

4. Calhoon, M., "Field Testing with the Pressure-Meter", Lecture given at the University of Kansas, 1970, U.S.A. (unpublished).

5. Gibson, R. E. and Anderson, W. F., "In-Situ Measurement of Soil Properties with the Pressure-Meter", Civil Engineering and Public Works Review, London, May, 1961.

6. Hartman, J. and Schmertmann, J., "FEM Study of Elastic Phase of Pressuremeter Test". Preceedings of the Specialty Conference on In-Situ Measurement of Soil Properties, American Society of Civil Engineers, Raleigh, North Calolina, 1975, pp. 190-207.

7. Laier, J. E., Schmertmann, J. H., "Effect of Finite Pressuremeter Length in Dry Sand", Proceedings of the Conference on In Situ Measurement of Soil Properties, American Society of Civil Engineers, Raleigh, N.C., 1975, Volume 1, pp241-259.

8. Lukas, G.L., Le Cler deBussy, B., "Pressuremeter and Laboratory Test Correlations for Clays", Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Volume 102, No. GT9, September, 1976, pp945-963.

9. Menard, L., "Interpretation and Application of Pressuremeter Test Results", Sols Soils, No. 26, 1975.

10. Menard, L. and B. E. Hartman, "Rules for the Calculcation and Design of Foundation Elements on the Basis of Pressuremeter Investigations in the Ground", Proceedings, 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, 1964.

11. Mori, H. and Tajima, S., "The Application of Pressuremeter Method to the Design of Deep Foundations", Soil and Foundations, February, 1964.

12. Roy, M., Juneau, L., LaRochelle, P., Tavenas, F., "In-Situ Measurement of the Properties of Sensitive Clays by Pressuremeter Tests", Proceedings of the Specialty Conference on In Situ Measurement of Soil Properties, American Society of Civil Engineers, Raleigh, North Carolina, 1975, Volume 1, pp. 350-372.

13. Schmertmann, J. H., "Measurement of In Situ Shear Strength", Proceedings of the Conference on In Situ Measurement of Soil Properties, American Society of Civil Engineers, Raleigh, 1975, Volume 2, pp 57-138.

Bearing Capacity Factor, K - a factor which varies according to the type of soil and increases with relative depth until a critical depth of embedment, H_c is reached. It is equal to 0.8 for footing foundations situated at the ground surface and can increase up to 9.0 for pile or pier foundations driven into very dense granular soils.

Creep - in a pressuremeter test, the reading of the deformations in relation to time are taken at 15 seconds, 30 seconds, and one (1) minute after pressure increase for each pressure step. The difference between 30 second and one minute readings is termed the "creep value"; it is used to help evaluate transition of the test from the pseudo-elastic to plastic phase, which defines the creep pressure, P_f.

Creep Pressure P_f - is the stress level at the end of the pseudo-elastic zone, before the soil goes into a plastic zone in which deformations are non-recoverable.

Lateral Earth Pressure At-Rest, P_o - is the coefficient of earth pressure at rest, K_o, times the vertical overburden pressure, $\gamma_d h$ at the elevation of the foundation element. It is sometimes evident from the pressuremeter curve when a flatter portion of the curve exists before the portion of the curve which defines the modulus of deformation, E_d.

Limit Pressure, P_l - is the pressure at which failure occurs for the soil. If the soil does not fail, there are several techniques to extrapolate the available data to estimate the limit pressure.

Modulus of Deformation, E_d - is a distortion modulus of the soil measured in the "pseudo-elastic" phase of the test. It is measured as the slope of the volume change versus pressure curve in the "straight-line" portion of the curve. The pseudo-elastic portion of the test is where the strains are partially recoverable.

Rheologic Coefficient - α, is a variable based on the structure of the soil. In general pressuremeter use they are used as follows:

sand & gravel	$\alpha = 1/3$
silt	$\alpha = 1/2$
clay	$\alpha = 2/3$
peat & newly compacted soil	$\alpha = 1.0$

Shape Coefficient - λ_2 and λ_3 derived from the formula of Steinbrenner for calculation of settlements by elastic theory. These coefficients are a function of the size relationships of a foundation as:

for round cross section	$\lambda_2 = 1$	$\lambda_3 = 1$
for square cross section	$\lambda_2 = 1$	$\lambda_3 = 1.1$
for rectangular cross sections $L/2_R = 2$,	$\lambda_2 = 1.53$,	$\lambda_3 = 1.2$
for rectancular cross sections $L/2_R = 10$,	$\lambda_2 = 2.65$,	$\lambda_3 = 1.5$

Settlement Components W: W_3 - is a settlement related to a phenomenon of compressibility. Directly under a foundation, the stress tensor has a spherical tendency; the increase in average pressure induces a volume reduction of the material in accordance with the value of the volumetric compression modulus.

W_2 - is a settlement related to a creep phenomenon under the influence of the deviatoric tensor, wherein, angular deformations are observed without volume change under the foundation.

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

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Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

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